OHIO DEPARTMENT OF TRANSPORTATION
CENTRAL OFFICE, 1980 W. BROAD ST., COLUMBUS, OHIO 43216-0899

July 18, 2008

To: Users of the Bridge Design Manual

From: Tim Keller, Administrator, Office of Structural Engineering

By: Sean Meddles, Bridge Standards Engineer

Re: 2008 Third Quarter Revisions

Revisions have been made to the ODOT Bridge Design Manual, July 2007. This package contains the revised pages. The revised pages have been designed to replace the corresponding pages in the book and are numbered accordingly. Revisions, additions, and deletions are marked in the revised pages by the use of one vertical line in the right margin. The header of the revised pages is dated accordingly.

To keep your Manual correct and up-to-date, please replace the appropriate pages in the book with the pages in this package.

To ensure proper printing, make sure your printer is set to print in the 2-sided mode.

The July 2007 edition of the Bridge Design Manual may be downloaded at no cost using the following link: http://www.dot.state.oh.us/se/BDM/BDM2007/bdm2007.htm

Attached is a brief description of each revision.

Summary of Revisions to the July 2007 ODOT BDM

BDM Section	Affected Pages	Revision Description
202.2.3.2.a	2-12	This change reflects the new terminology for rock classification (e.g. soft rock = weak rock, hard rock = strong rock) in the ODOT Specifications for Geotechnical Exploration, Jan. 2007
208	2-36 through 2-37.2	This change clarifies the design requirements for temporary support of existing structures.
302.2.3	3-14 through 3-14.2	This section has been expanded to include new plan elevation requirements. Screed Elevations have been redefined and are no longer required above all beam/girder lines. This was done to avoid confusion with the new Top of Haunch Elevation. The introduction of the Top of Haunch Elevation above each beam/girder simplifies the contractor's deck falsework calculations. The availability of Final Deck Elevations simplifies the troubleshooting process for deck placement issues should these arise.
302.2.7	3-16 through 3-17.12	This new section addresses the deck placement issues that were presented in our office's seminar, "The State of Practice for Highly Skewed Bridges".
302.4.1.14 & Fig. 302.4.1.14-1	3-25	With the retirement of BS-1-93, a common plan error occurring for bolted splice designs is the interference of splice plates and rolled beam fillets. This figure was created to illustrate the practical limits for the splice plates in order to avoid beam fillets.
302.4.1.14.a	3-25 through 3-25.2	This change addresses the proper bolt finishes required for partially painted weathering steel beams and girders. If both faying surfaces for bolts are not coated, then the contact between the dissimilar metal coatings (i.e. galvanizing vs. weathering steel) will create a galvanic cell causing early deterioration of the coating.
303.4.2.6	3-65	This change reflects a revision to pile restrike requirements in the C&MS 523.02 which specifies a restrike item to consist of performing dynamic testing on two piles with a CAPWAP analysis on one of the two piles tested. Previous editions of the C&MS did not clearly define these restrike requirements.
303.4.2.7	3-66	Refer to the BDM Section 303.4.2.6 revision description.
303.4.2.8	3-66 through 3-68	Refer to the BDM Section 303.4.2.6 revision description.
602.3	6-4	This change corrects a material discrepancy with the C&MS 711.03.

BDM Section	Affected Pages	Revision Description
606.1	6-11 through 6-13	Changes to notes [606.1-1], [606.1-2] and [606.1-3] allow the contractor to perform dynamic testing to determine refusal in lieu of the traditional 20 bpi criteria avoiding excessive wear and tear on the driving equipment.
606.2	6-15	Refer to the BDM Section 303.4.2.6 revision description.
610.7	6-25 through 6-26	This new section addresses the deck placement issues that were presented in our office's seminar, "The State of Practice for Highly Skewed Bridges".
702.14	7-11 through 7-12.2	This section has been revised to comply with the revisions to BDM Section 302.2.3
Fig. 702.14-1		This figure has been revised to comply with the revisions to BDM Section 302.2.3
Fig. 702.14-2		This figure has been revised to comply with the revisions to BDM Section 302.2.3

calculated in accordance with Section 203.3(D), should be considered in the design of the substructures; the location of the bottom of footings; the minimum tip elevations for piles and drilled shafts; and the factored side resistance of piles and drilled shafts. See BDM Section 202.2.3.2.h for more information.

Where downdrag has been identified as a potential contributor to the total factored load, the estimated downdrag load shall be included in the report. See BDM Section 202.2.3.2.c for more information.

The Foundation Report for MSE wall supported abutments shall include calculations for external stability (*LRFD 11.10.5*) and settlement. The report shall also consider the effect of settlement and include all construction constraints, such as soil improvement methods, that may be required.

Specific design considerations for each foundation type are presented in the following sections.

202.2.3.1 SPREAD FOOTINGS

The use of spread footings shall be based on an assessment of the following: design loads; depth of suitable bearing materials; ease of construction; effects of flooding and scour analysis; liquefaction and swelling potential of the soils; frost depth; and amount of predicted settlement versus tolerable structure movement.

Spread footings shall be designed in accordance with *LRFD 10.6*.

Elevations for the bottom of the footing shall be shown on the Final Structure Site Plan. The estimated size of the footing; estimated settlements; and the factored bearing resistances shall be provided for review with the Foundation Report.

Adjust the footing size, the amount of predicted settlement and the factored bearing resistance during detail design as the design loads for the Service, Strength and Extreme Event Limit State are refined.

All spread footings at all substructure units, not founded on bedrock, are to have elevation reference monuments constructed in the footings. This is for the purpose of measuring footing elevations during and after construction for the purpose of documenting the performance of the spread footings, both short term and long term. See Section 600 for notes and additional guidance.

202.2.3.2 PILE FOUNDATIONS

Pile foundations should be considered when spread footing foundations are prohibited or are not feasible.

The type, size and estimated length of the piles for each substructure unit shall be shown on the Final Structure Site Plan. The estimated length for piling shall be measured from the pile tip to the cutoff elevation in the pile cap and shall be rounded up to the nearest five feet [one meter]. To determine the estimated length for different pile types, refer to BDM Section 202.2.3.2.a and 202.2.3.2.b. The estimated length may need to be adjusted during detail design as the design loads for the Service, Strength and Extreme Event Limit States are refined.

202.2.3.2.a PILES DRIVEN TO REFUSAL ON BEDROCK

When piles are driven to refusal on the bedrock, the plans should specify steel ‘H’ piles.

Refusal is met during driving when the pile penetration is an inch or less after receiving at least 20 blows from the pile hammer. When estimating pile length, the depth to refusal shall be assumed as the elevation on the nearest soil boring where the rock core begins.

The total factored load ($\sum \eta_i \gamma_i Q_i$) for each pile shall be provided in the structure general notes. A sample note is provided in BDM Section 600. The plan specified value for total factored load shall be the factored load for the highest loaded pile at each substructure unit.

The factored resistance for piles driven to refusal on bedrock is typically governed by structural resistance. The total factored load for any single pile shall not exceed the maximum factored structural resistance ($R_{R\ max}$). The commonly used H-pile sizes and the maximum factored structural resistance ($R_{R\ max}$) allowed for each are listed below:

H Pile Size	$R_{R\ max}$
HP10X42	310 kips
HP12X53	380 kips
HP14X73	530 kips

H Pile Size	$R_{R\ max}$
HP250X62	1380 kN
HP310X79	1690 kN
HP360X108	2360 kN

The values listed above for the maximum factored structural resistance assume: an axially loaded pile with negligible moment; no appreciable loss of section due to deterioration throughout the life of the structure; a steel yield strength of 50 ksi; a structural resistance factor for H-piles subject to damage due to severe driving conditions (*LRFD 6.5.4.2*: ϕ_c = 0.50); and a pile fully braced along its length. These values should not be used for piles that are subjected to bending moments or are not supported by soil for their entire length. Examples include piles for capped pile piers and piles in soils subject to scour.

For piers, other than capped pile piers, the preferred H-pile size is HP10X42 [HP250X62]. For information regarding piles for capped pile piers, refer to BDM Section 303.3.2.5.

In order to protect the tips of the steel “H” piling, steel pile points shall be used when piles are driven to refusal onto strong bedrock. When the depth of overburden is more than 50 feet [15 meters] and the soils are cohesive in nature, piles driven to strong bedrock generally should not

Volume Three, as projects that do not alter the basic highway cross section or geometry, require no additional right-of-way, are exempt from Categorical Exclusion documentation, and require little or no public involvement. Minimal project types include: bridge painting, deck overlays, scupper installations, barrier facings, concrete sealing, partial depth concrete repairs, etc. Minimal projects do not require a preliminary design submission.

207 BRIDGE GEOMETRICS

207.1 VERTICAL CLEARANCE

The “Required Minimum” and “Actual Minimum” Vertical Clearances and their locations shall be shown on the Preliminary Structure Site Plan, Section 201.2.2. The “Actual Minimum” Vertical Clearance is the minimum overhead clearance provided by the design plans. For new grade separation structures, the “Required Minimum” Vertical Clearance shall not be less than the preferred clearance specified in ODOT’s Location and Design Manual, Figure 302-1 unless otherwise specified in the scope of services. A “Required Minimum” Vertical Clearance less than the L&D Manual minimum clearance will require a Design Exception in accordance with Section 105 of the L&D Manual.

207.2 BRIDGE SUPERSTRUCTURE

Bridge superstructure widths shall be established in accordance with ODOT's Location and Design Manual, Section 302, unless specified in the scope of services or other contract criteria.

207.3 LATERAL CLEARANCE

Divided highways having four or more lanes crossing under an intersecting highway shall be provided with a minimum lateral clearance of 30 feet [9000 mm] from the edge of traveled lane to the point where the 2:1 back slope intersects the radius at the toe of the 2:1 slope. Refer to ODOT's Location and Design Manual, Figure 307-2. To satisfy cost considerations or in order to maintain the typical roadway section (including roadway ditch) of the underpass through the structure, for four or more lane highways, wall abutments or the 2:1 slope of typical two-span grade separation structures may be located farther than 30 feet [9000 mm] from the near edge of traveled lane.

Lateral clearances for other roadway classifications shall be established in accordance with ODOT's Location and Design Manual, Section 302, unless specified in the scope of services or other contract criteria.

207.4 INTERFERENCE DUE TO EXISTING SUBSTRUCTURE

Where a new pier or abutment is placed at the location of an existing pier or abutment the usual

"Removal" note (and also the text of CMS 202.03) calls for sufficient removal of the old pier or abutment to permit construction of the new. However, a new pier or abutment preferably should not be located at an existing pier or abutment where the existing masonry may extend appreciably below the bottom of the proposed footing, or appreciably below the ground in case of capped-pile construction. This applies particularly where piles are to be driven. It is desirable to avoid the difficulty and expense of removing deep underground portions of the existing substructure and to avoid the resultant disturbance of the ground.

Where existing substructure units are shown on the Site Plan, the accuracy of the locations and extent should be carefully drawn. The existing substructure configuration should be shown based on existing plans or field verified dimensions, otherwise just a vertical line showing the approximate face of the abutment or pier widths should be shown. Misrepresentation of the location of the existing substructure units has resulted in expensive change orders during construction. Existing dimensions should be labeled as (+/-) plus or minus.

207.5 BRIDGE STRUCTURE, SKEW, CURVATURE AND SUPERELEVATION

During the Assessment of Feasible Alternatives, the location of the proposed structure should be studied to attempt to eliminate the presence of excessive skew, curves or extreme superelevation transitions within the actual bridge limits.

208 TEMPORARY SHORING

208.1 SUPPORT OF EXCAVATIONS

Whenever shoring is required to support a roadway where traffic is being maintained and the height of the retained earth will be over eight feet [2.5 meters], the Design Agency shall be required to provide a temporary shoring design with details provided in the plans and feasibility studied during the Structure Type Study.

For projects involving Railroads, the requirements will be different as each railroad company has their own specific requirements. The Design Agency is responsible for contacting the responsible railroad and obtaining the specific requirements for design and construction.

Following are some conceptual ideas for the design of temporary shoring:

A. A cantilever sheet pile wall should generally be used for excavation up to approximately 12 feet [3.5 meters] in height. Design computations are necessary.

B. For cuts greater than 12 feet [3.5 meters] in height, anchored or braced walls will generally be required.

C. For anchored walls, the use of deadmen is preferred. Braced walls using waler and struts can sometimes be braced against another rigid element on the excavated side. The use of soil or rock anchors(tiebacks) is generally the last option considered in the design of anchored walls.

D. The use of steel “H” piles with lagging is also a practical solution for some sites. Please note that some railroad companies allow only interlocking steel sheet piling adjacent to their tracks.

E. Where sufficient embedment cannot be attained by driving sheet piling because of the location of shallow bedrock, predrilled holes into the bedrock with soldier “H” piles and lagging should be considered.

   For cuts greater than 12-15 feet [3.5-4.5 meters], the “H” piles may need to be anchored.

F. The highway design live loading should be equal to two feet [600 mm] of equivalent soil height as a surcharge.

G. The following items at a minimum should be shown on the detail plans:

   1. Minimum section modulus
   2. Top and minimum bottom elevation of shoring
   3. Limits of shoring
   4. Sequence of installation and/or operations.
   5. Method of payment
   6. If bracing or tiebacks are required, all details, connections and member sizes shall be detailed.
   7. A general note in plans allowing a Contractor designed alternate for temporary shoring.

208.2 SUPPORT OF EXISTING STRUCTURE

Whenever temporary support is required for a portion of an existing structure used to maintain traffic, the Design Agency shall provide sufficient information in the plans to allow contractors to prepare bids and construct the project. The feasibility of temporary support of an existing structure should be considered and discussed during the Structure Type Study.

The design shown in the plans should include: permissible locations of temporary support; temporary support loads; construction sequences; construction limitations not otherwise provided in C&MS 501.05; and any remaining plan notes. As a minimum, the plan notes should address method of measurement and basis of payment for temporary support.

209 MISCELLANEOUS

209.1 TRANSVERSE DECK SECTION WITH SUPERELEVATION

If the change in cross slope at the superelevation break point is less than or equal to 7 percent, then no rounding is required. For changes greater than 7 percent the bridge deck surface profile shall be as follows:

A. When the roadway break point is located between roadway lanes (not at the edge of pavement) the bridge cross slope is to extend to the toe of parapet. See “CASE a” in Figure 209.1-1.

B. When the roadway break point is located at the edge of pavement (adjacent shoulder width is less than four feet [1.2 meters]), the bridge cross slope is to be continued past the break point to the toe of deflector parapet. See “CASE b” in Figure 209.1-1.

C. When the roadway break point is located at the edge of pavement (adjacent shoulder width is equal to or greater than four feet [1.2 meters] and less than eight feet [2.4 meters]), a four foot [1.2 meter] rounding distance from the edge of pavement onto the shoulder is used to transition from the bridge cross slope to the 0.5 in. per ft. [0.04] shoulder cross slope. See “CASE c” in Figure 209.1-2.

D. When the roadway break point is located at the edge of pavement (adjacent shoulder width is equal to or greater than eight feet [2.4 meters]), a five foot [1.5 meter] rounding distance

(THIS PAGE INTENTIONALLY LEFT BLANK)

from the edge of pavement onto the shoulder is used to transition from the bridge cross slope to the 0.5 in. per ft. [0.04] shoulder cross slope. See “CASE d” in Figure 209.1-2.

The transition from the roadway approach transverse section to the bridge deck transverse section is to take place within the limits of the approach slab, whenever possible. On bridges with high skews, it may not be possible to do the transition within these limits and other alternatives should be considered during the Assessment of Feasible Alternatives.

For decks with over the side drainage, the treatment of the deck and the shoulder slopes shall be as described in subsections a through d above except that the slope shall continue to the edge of the deck.

209.1.1 SUPERELEVATION TRANSITIONS

Because of the complexities associated with superelevation transitions on bridge superstructures (i.e. beam and girder cambering, crossframe fabrication, deck form construction, slip forming of parapets, etc.) all reasonable attempts should be made to keep such transitions off of bridge decks. Where transitions must be located on bridge decks, preferably, the transitions should be straight. An example of a transition diagram is shown in Figure 209.1.1-1. A table with the information shown in Figure 209.1.1-1 is also acceptable. Where this is not practicable, then transition’s discontinuities should be smoothed by inserting 50 foot [15 meter] roundings at each discontinuity.

209.2 BRIDGE RAILINGS

All bridge structures on the National Highway System (NHS) or the State System require the use of crash tested railing meeting the loading requirements of TL-3 as defined by NCHRP report 350. The requirement for the NHS became effective October 1, 1998. For detailed information, refer to Section 304.

For structures with over the side drainage on the National Highway System, Twin Steel Tube Bridge Guardrail, Standard Bridge Drawing TST-1-99 should be used.

Over the side drainage shall not be used for bridges over highways and railroads. For four lane divided highways concrete deflector parapets shall be used. For bridges with heights of 25 feet [7.6 meters] or more above the lowest groundline or normal water, concrete deflector parapets should be used.

Refer to Section 305 of this Manual for vandal protection fencing requirements.

209.3 BRIDGE DECK DRAINAGE

The preferred minimum longitudinal grade of the bridge deck surface, when using concrete parapets, is 0.3 %, whenever possible.

(THIS PAGE INTENTIONALLY LEFT BLANK)

(THIS PAGE INTENTIONALLY LEFT BLANK)

provided with Figure 302.2.2-1.

Transverse spacing of the top and bottom reinforcing in a deck design shall meet section 302.2.4.2.

302.2.3 DECK ELEVATION REQUIREMENTS

302.2.3.1 SCREED ELEVATIONS

Screed elevations are control elevations for concrete deck finishing machines that account for dead load deflections to ensure that the bridge deck is completed to the correct elevation. To establish screed elevations, the final surface elevations are adjusted for non-composite deflections resulting from deck placement and composite deflections resulting from utility and railing loads. Screed elevations shall not include adjustment for deflections due to the future wearing surface loading. Calculated deflections caused by the weight of the deck concrete should assume a completed placement sequence. Use deflection data from girder lines closest to each screed line to determine elevations. Refer to Figure 302.2.3-1.

If the deflections are determined through a line girder analysis method, the deck load should be distributed evenly to all beams/girders loaded in each construction phase to establish screed elevations. If a refined analysis method is used, establish screed elevations using the individual beam/girder deflections.

The bridge plans shall include a screed elevations table. The locations of all screed elevations in the table should be identified on a transverse section and plan view. Elevations should be provided for all: curblines or deck edges; profile grade points; transverse grade-break lines; and phased construction lines for the full length of the bridge. Screed elevations are not required above beam/girder lines. Bearing points, quarter-span points, mid-span points and splice points shall be detailed as well as any additional points required to meet a maximum spacing between points of 25'-0" [7.5 m].

For bridges with a separate wearing course, the elevations given should be those at the top of the portland cement concrete deck. Provide a plan note stating at what surface the elevations are given in order to eliminate any confusion.

Screed elevations are not required for non-composite box beam bridges or slab bridges. Screed elevations for composite box beam bridges shall meet the same requirements as steel beam, girder and prestressed I-beam bridges.

302.2.3.2 TOP OF HAUNCH ELEVATIONS

Top of haunch elevations represent the theoretical bottom of deck elevation before the concrete deck is placed. Top of haunch elevations should be provided at the centerline of each girder at bearing points, quarter points, mid-span points, splice points and additional points to meet a maximum spacing between points of 25'-0" [7.5 m]. The top of haunch elevation locations should be identified in a plan view and on the transverse section. Top of haunch elevations are not required for composite box beam bridges. Provide a plan note for a definition and description of the purpose for the top of haunch elevations (see BDM Section 700). Refer to Figure 302.2.3-1.

302.2.3.3 FINAL DECK SURFACE ELEVATIONS

Final deck surface elevations represent the position of the deck after all dead loads except future wearing surface have been applied. Final deck surface elevations shall be provided at bearing points, quarter points, mid-span points, splice points and additional points to meet a maximum spacing between points of 25'-0" [7.5 m] for each: girder centerline; curbline or deck edge; transverse grade-break line; and phased construction line. The final deck surface elevation locations should be identified in a plan view. Refer to Figure 302.2.3-1.

302.2.4 REINFORCEMENT

302.2.4.1 LONGITUDINAL

Secondary reinforcement in the top-reinforcing layer of a reinforced concrete deck on steel or concrete stringers shall be approximately 1/3 of the main reinforcement, uniformly spaced.

Research has shown that secondary bars in the top mat of reinforced concrete bridge decks on stringers should be small bars at close spacing. Therefore the required secondary bar size shall be a #4 [#13M]. The only exception to this requirement is if the bar spacing becomes less than 3 inches [75 mm].

(THIS PAGE INTENTIONALLY LEFT BLANK)

For stringer type bridges with reinforced concrete decks, the secondary bars shall be placed above the top of deck primary bars. This helps in reducing shrinkage cracking and adds additional cover over the primary bars.

For reinforced concrete deck slabs on non-composite stringer type bridges, where the main reinforcement is transverse to the stringers, additional top longitudinal reinforcement shall be provided in the negative moment region over the piers. This additional secondary reinforcement shall be equal to the distributional reinforcement (1/3 of the main reinforcement). This additional reinforcement shall be uniformly spaced and furnished in length equal to the larger of: 40 percent of the length of the longer adjacent stringer span or a length that meets the requirements of *LRFD 5.11.1.2.3*.

For composite designs, the total longitudinal reinforcement over a pier shall meet the requirements of *LRFD 6.10.1.7*.

Additional negative moment reinforcement should be placed approximately symmetrical to the centerline of pier bearings but with every other reinforcing bar staggered 3 feet [1000 mm] longitudinally.

302.2.4.2 TRANSVERSE

To facilitate the placement of reinforcing steel and concrete in transversely reinforced deck slabs top and bottom main reinforcement shall be equally spaced and placed to coincide in a vertical plane.

For steel beam or girder bridges with a skew of less than 15 degrees the transverse reinforcing may be shown placed parallel to the abutments. Bridges with a skew greater than 15 degrees or where the transverse reinforcing will interfere with the shear studs should have the transverse reinforcement placed perpendicular to the centerline of the bridge. Refer to the appropriate Standard Bridge Drawing for the requirements on slab bridges.

For prestressed I-beams, transverse reinforcing shall be placed perpendicular to the centerline of the bridge.

For composite box beam decks, the transverse reinforcing steel may be placed parallel to the abutment.

For steel beam or girder bridges, the clearance of the bottom transverse bars over the top of any bolted beam splice plates or moment plates should be checked as reinforcing bars at a skew generally cannot be placed between bolt heads.

302.2.5 HAUNCHED DECK REQUIREMENTS

Concrete decks on steel beam, girder or prestressed I-beam structures shall have a concrete haunch to prevent a thinning of the deck slab as a result of unforeseen variations in beam camber. At a minimum, the design haunch shall allow for 2 inches [50 mm] of excessive camber. For steel beam and girder structures, the haunch shall be tapered back to the original concrete deck thickness in a 9 inch [225 mm] length and the concrete haunch shall encase the edges of the top flange. See Figures 302.2.5-1 & 302.2.5-2.

302.2.6 STAY IN PLACE FORMS

Galvanized steel or any other material type, stay in place forms, shall not be used.

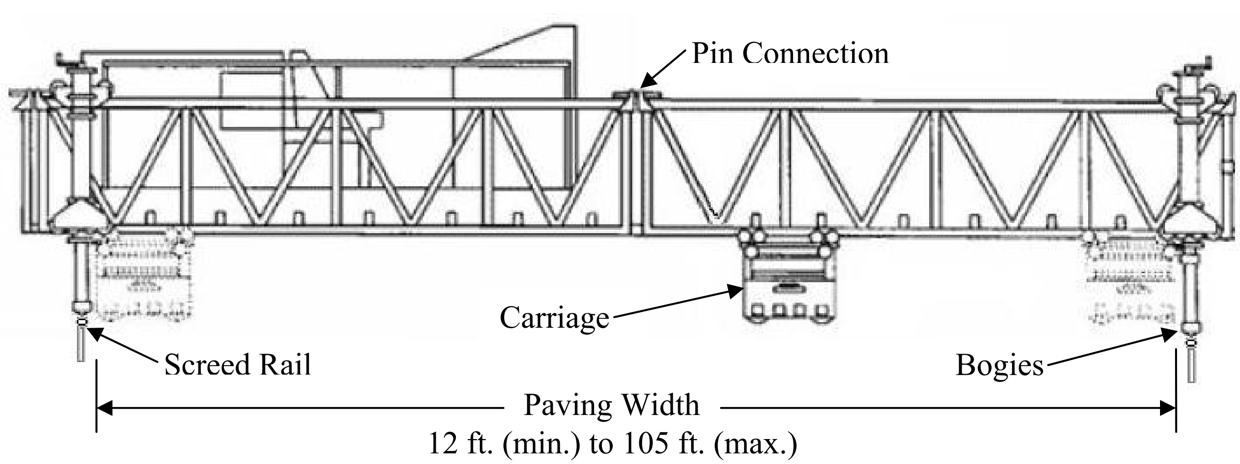
302.2.7 CONCRETE DECK PLACEMENT CONSIDERATIONS

Mechanized finishing machines are preferred to hand finishing methods for both consistency of surface finish and economics. Designers should be aware of finishing machine limitations in order to avoid deck designs that require hand finishing methods.

The placement of deck concrete using mechanized finishing machines alone does not ensure a smooth riding surface. Achieving a smooth riding surface as well as ensuring the proper geometry of the concrete deck is further complicated by deflections of the concrete falsework and of the main structural support members during the placement operation. The Contractor is responsible for designing falsework and finishing machine support to minimize deflection during placement, but the Designer is responsible for deflections induced by deck placement on the superstructure. Many complications due to deflection during placement can be avoided with proper design considerations.

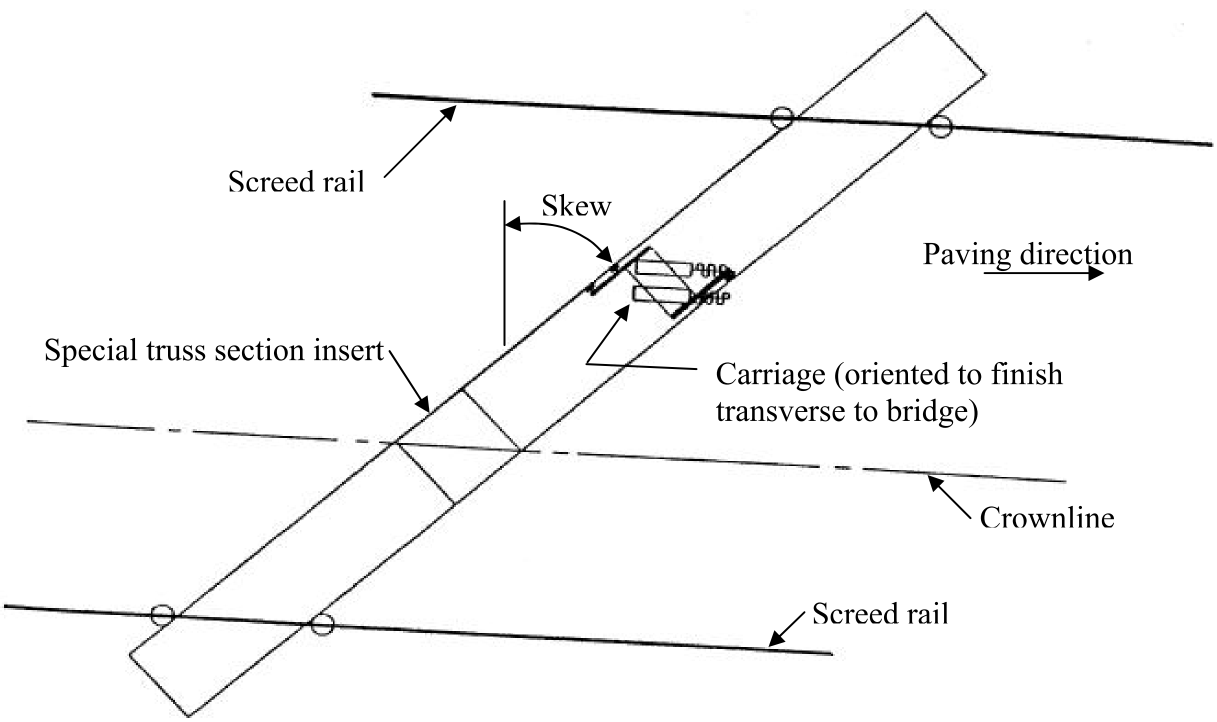
302.2.7.1 FINISHING MACHINES

Mechanized finishing machines are comprised of fabricated truss sections pinned together to span the bridge deck width to be paved. The truss spans are supported at each end on a set of wheels, called “bogies,” which ride along the length of the bridge on screed rails. Suspended below the truss is a finishing head, called a “carriage,” which levels, compacts, vibrates and finishes the concrete.



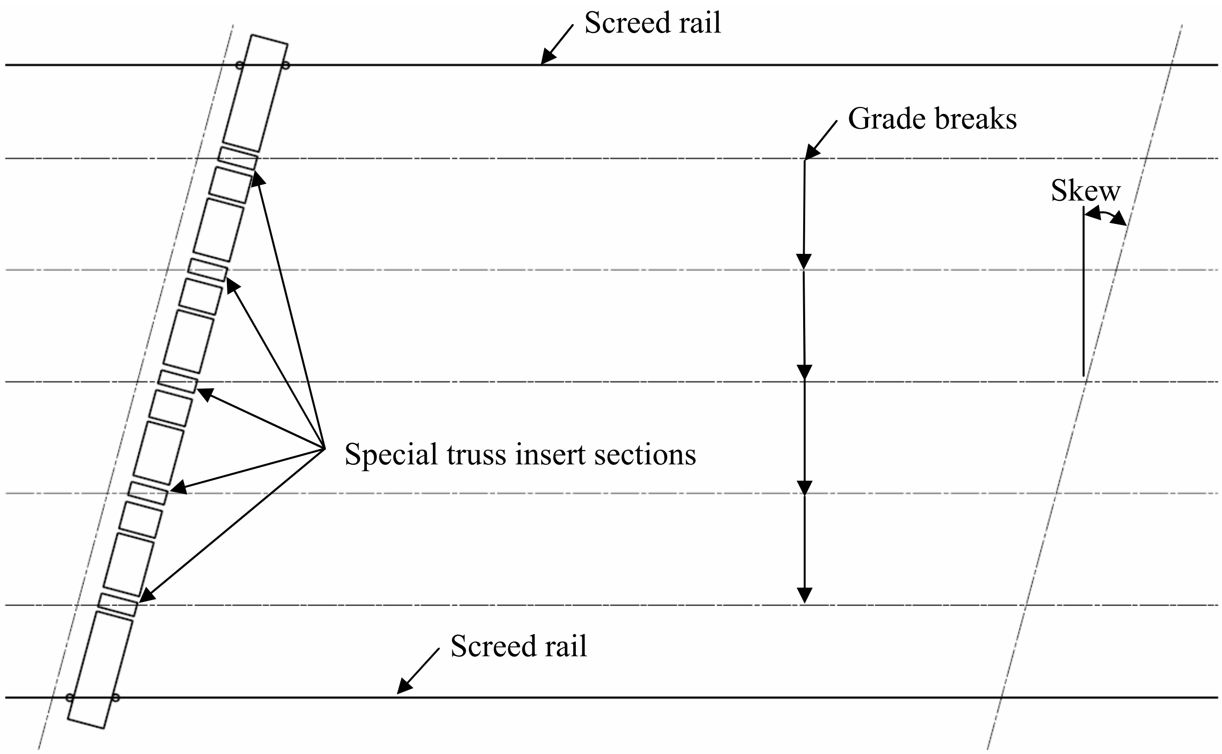
Finishing machines can be placed such that the truss sections are skewed with respect to the screed rails. This orientation allows for concrete placement parallel to the substructure skew as required by the C&MS 511. For skew angles of 15º and greater, the finishing machine can be skewed to within 5 degrees of the plan specified skew angle.

The carriage can also be skewed with respect to the truss sections. This feature allows the carriage to finish the concrete transverse to the bridge when the truss sections are placed at some other orientation (e.g. parallel to the substructure skew). In order to ensure a proper finish at transverse grade breaks (e.g. crown points), the carriage should always be oriented to finish the concrete transverse to the bridge. A special length truss section insert is required above the grade break locations such that the grade break line lies directly below opposite corners of the section. For skewed bridges without transverse grade breaks, skewing the carriage with respect to the truss sections is not required.

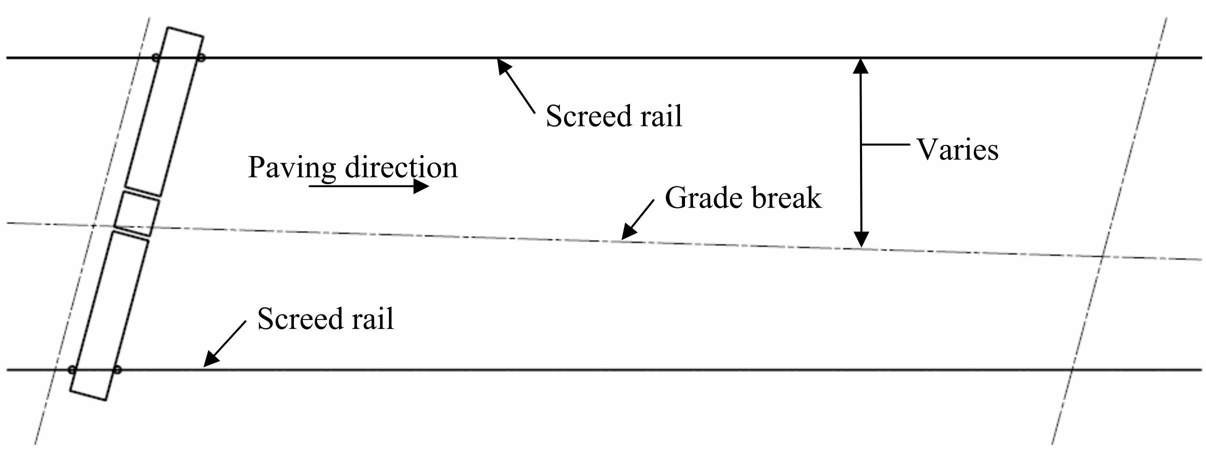


Most finishing machines do not easily accommodate non-parallel rails. The distance between the screed rails should be a fixed width. Designs that require tapered paving widths should be avoided.

The finishing machines can be hinged at the pin connections between truss sections in order to provide transverse grade breaks (e.g. crown points). In theory, multiple transverse grade breaks can be accommodated, but the grade breaks must remain at a fixed spacing in order to line up with a pin connection. The figure below illustrates the complexity of the machine set-up to accommodate multiple grade breaks in a transverse section placed on a skew. Note that the length of truss sections required between grade breaks must fit the standard truss section lengths.



Grade break locations that move laterally along the length of the bridge cannot be paved in a single operation using a mechanized finishing machine and should therefore be avoided. Note that as the machine progresses forward, the truss hinge locations and the grade break locations no longer coincide. See the figure below.



302.2.7.2 SOURCES OF GIRDER TWIST

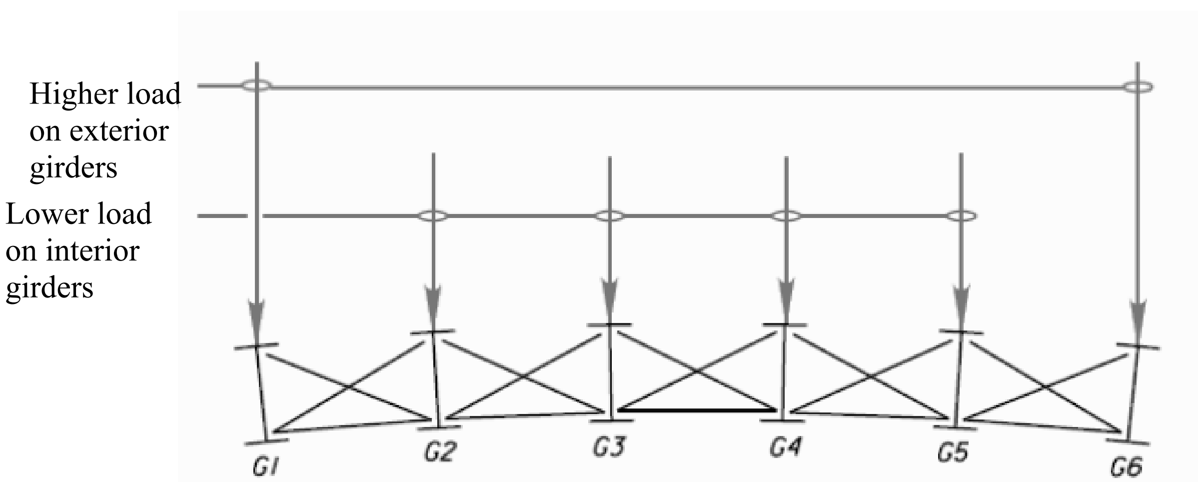
The interconnectivity between girders, intermediate crossframes/diaphragms and end crossframes/diaphragms is essential to a structure's stability throughout the construction process. Therefore, it is of utmost importance to ensure that all crossframes/diaphragms are fully installed prior to deck placement. Failure to do so may lead to construction disputes, expensive repairs and lengthy construction delays or even impact project safety. One major drawback to this interconnectivity is that the deflection caused by the placement of the concrete deck will result in girder twisting.

There are primarily three independent sources of girder twist resulting from deck placement. This manual will refer to these sources as: global superstructure distortion, oil-canning and girder warping.

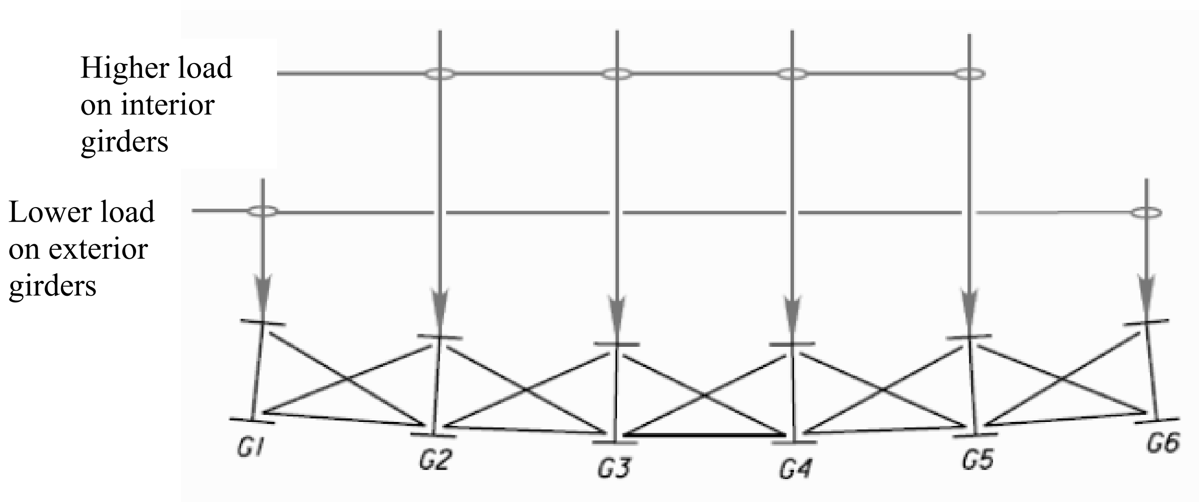
302.2.7.2.a GLOBAL SUPERSTRUCTURE DISTORTION

Global superstructure distortion is distortion of the bridge transverse section primarily caused by differential deflections between adjacent girders. As a girder deflects downward with respect to an adjacent girder, the rigidity of the cross framing between the two girders causes the deflecting girder to rotate as it deflects. This distortion may occur with both steel and prestressed concrete superstructures. The most common differential deflections occur between the exterior girders and adjacent interior girders for a given construction phase when the loaded tributary areas over the girders differ.

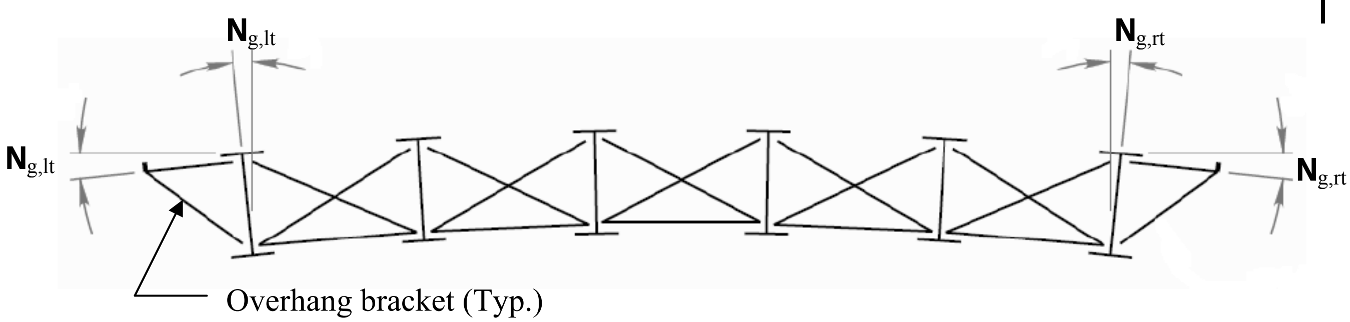
Transverse sections with more heavily loaded exterior girders distort in a convex shape.



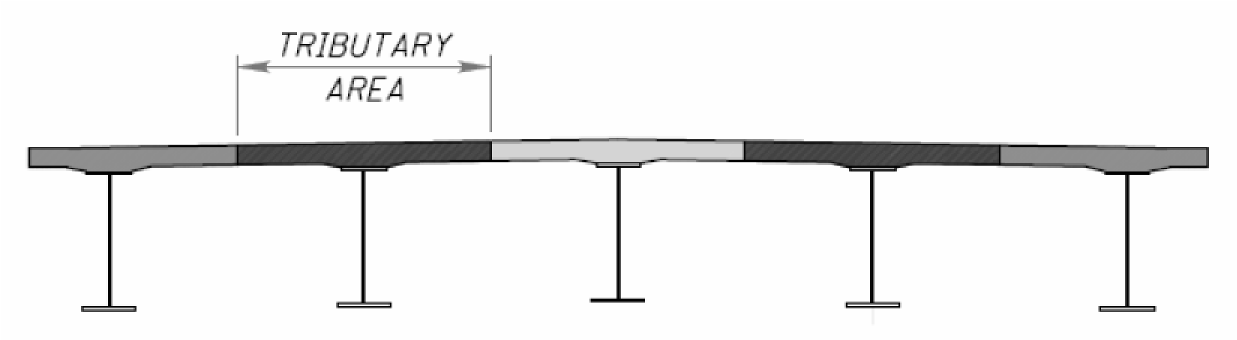
Transverse sections with more heavily loaded interior girders distort in a concave shape.



Twisting of the exterior girders can result in deck thickness and cover loss if the screed rails are supported on cantilevered falsework. The magnitude of girder twist (measured as $\mathbf{N}_g$) will vary over the length of the bridge and will be different for the left and right sides if loading or geometry is not symmetrical.



For bridges with tangent alignments and adjacent substructure skews that vary by no more than 15º, the magnitude of the girder twist can be reduced by utilizing transverse sections with balanced tributary deck loadings. For a new superstructure, the amount of girder twist due to global superstructure deformation can be neglected when the tributary deck load carried by the fascia girder does not exceed 110% of the average of the tributary deck load carried by the interior members for a given construction phase.

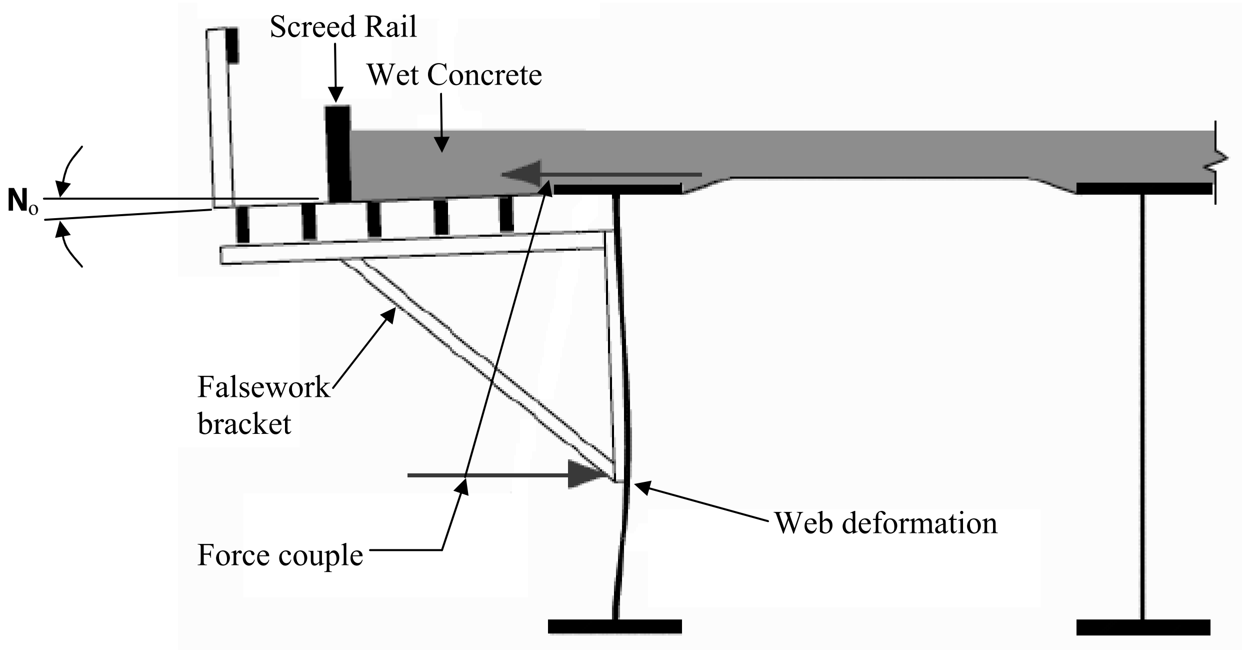


When the aforementioned tributary deck loading requirements of the fascia members cannot be met or, because of geometry, do not apply, the Designer shall perform a refined analysis of the superstructure system to determine the magnitude of fascia girder twist ($\mathbf{N}_g$) due to deck concrete placement. To properly calculate the effect of the twist angle on deck thickness, the analysis should be based on the deflection occurring due to the concrete present at the time that the finishing machine passes over the point under consideration. This degree of precision requires a separate refined analysis for each point of consideration. It is generally sufficient to calculate $\mathbf{N}_g$ based on the full wet concrete load placed over the entire structure. However, on complex structures with variable skews and/or curved girders, a higher degree of precision may be warranted to ensure proper deck thickness.

Additional measures to reduce global deformation include: adding or stiffening the crossframes/diaphragms; and increasing the stiffness of the girders. An increase in the crossframe stiffness results in better load distribution across the width of the structure and less distortion. An increase in the stiffness of the girders reduces the magnitude of vertical deflection resulting in less distortion of the transverse section.

302.2.7.2.b OIL-CANNING

Distortion due to oil-canning occurs when large lateral loads from the cantilevered deck slab falsework bracket deform the girder web.



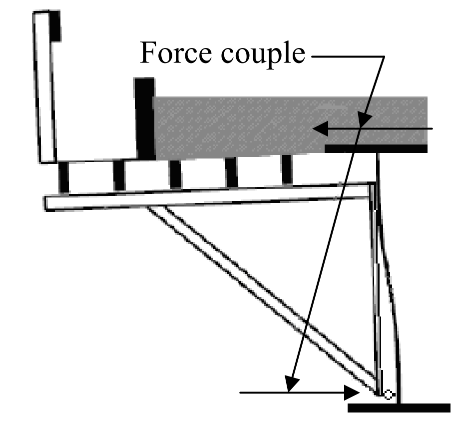
Locating the falsework bracket near the bottom flange will reduce the amount of web deformation. C&MS Item 508 requires the lower point of contact to be within 8” of the top of the bottom flange. Given this requirement and the geometric capabilities of the falsework brackets, the magnitude of girder twist (**N**$_o$) resulting from oil-canning may be neglected for girder webs 84” deep or less.

For web depths greater than 84”, designers shall provide the location of the falsework bracket in the plans. Provide a General Note that removes the lower point of contact requirement of C&MS Item 508 (see BDM Section 600 for an example). The pay item for deck concrete shall be “as per plan”. Using the plan bracket location, designers shall determine **N**$_o$. Designers may assume the lowest location of the falsework bracket to be 76” measured below the bottom of the top flange. The magnitude of twist can be predicted using finite element analysis of the web or by various approximate methods. If the magnitude results in excessive deck thickness loss, reducing the transverse stiffener spacing or adding temporary bracing on the inside of the web may be necessary. Any temporary bracing should be detailed in the plans.

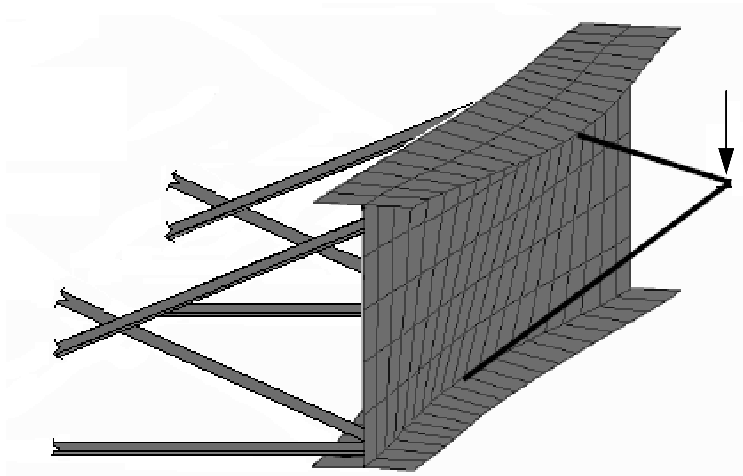
The magnitude of girder twist resulting from oil-canning may be neglected for prestressed I-beam superstructures.

302.2.7.2.c GIRDER WARPING

Distortion due to girder warping occurs as a result of deck slab overhang falsework loading on the fascia girder between points of lateral bracing (e.g. crossframes). The bracket loads produce twist between the crossframes due to a combination of girder warping and pure torsional distortion. The girder is restrained from warping at the crossframe locations. Due to the inherent torsional stiffness of prestressed I-beams, the distortion due to girder warping may be neglected. Other design considerations for I-beams due to the overhang bracket loadings are presented at the end of this section.



Girder warping



Torsional distortion

For steel superstructures, Designers should calculate the magnitude of twist (**N**$_w$) due girder warping using the TAEG software developed by the Kansas Department of Transportation. TAEG ("Torsional Analysis of Exterior Girders") is available at no cost and can be downloaded at: http://www.ksdot.org/kart/.

Since most of the data input in TAEG is dependent upon the contractor's equipment and falsework design, designers should use conservative assumptions to accommodate most contractor resources. For design-build projects and value engineering change proposals (VECP's), data input for TAEG shall represent the actual falsework and equipment to be used by the contractor. Designers may use the following assumptions in lieu of actual contractor supplied information:

A. Girder Data:

   For bridges with constant web depths, designers may select the cross section with the least torsional resistance to represent the entire structure. For bridges with variable depth webs, designers may disregard the effect of girder warping in the web depth transition sections.

B. Bridge Lateral Data:

   Designers may select the largest crossframe spacing to represent the entire structure. For structures with variable beam spacings (i.e. flared girders) designers may select the largest spacing dimension to represent the entire structure. Designers should generally avoid utilizing temporary tie rods and timber blocks; however, if required these should be detailed in the contract plans.

C. Permanent Lateral Support Data:

The default crossframe type assumed by the TAEG software consists of a stiffener and diagonal x-bracing with top and bottom horizontal chords. In order to analyze the structure with a standard ODOT crossframe, designers should input stiffener dimensions and select the "Diaphragms (Inputted Ix)" option. For ODOT Type 1 crossframes, designers should assume a fictitious stiffener of dimensions: 5" x 3/8". Determine the diaphragm moment of inertia for all standard ODOT crossframes as follows:

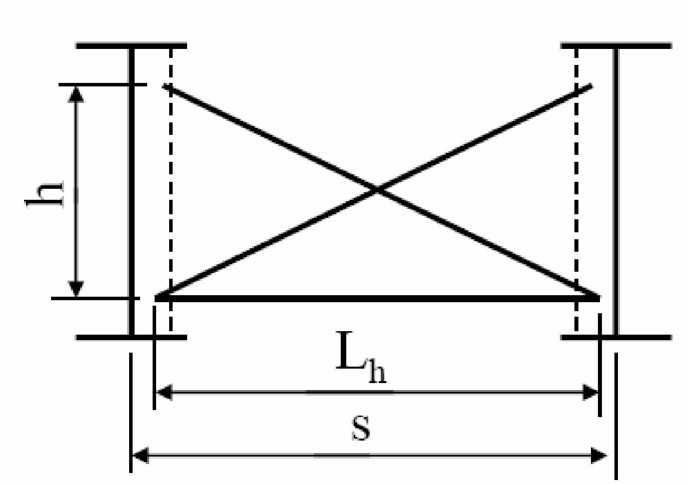
$$I_x = \frac{h^2 s}{4L_d^3\left(\frac{1}{A_d L_h^2} + \frac{L_h}{A_h L_d^3 + A_d L_h^3}\right)}$$

Where:

A_d = Area of the diagonal member (in^2)

A_h = Area of the horizontal member (in^2)

$$L_d = \sqrt{L_h^2 + h^2}$$



D. Temporary Lateral Support Data:

Designers should generally avoid utilizing temporary tie rods and timber blocks; however, if required these should be detailed in the contract plans.

E. Load Data:

1. Live Load on Walkway.......50 lb/ft^2
2. Live Load on Slab.......50 lb/ft^2
3. Dead Load of Formwork.......10 lb/ft^2
4. Dead Load of Concrete....... 150/t_{avg} lb/ft^2
   (t_{avg} = Average thickness [ft.] of deck slab overhang)
5. Wheel Spacing [1-2-3]....... 36" – 31" – 36"
6. Maximum Wheel Load:

   To estimate the total finishing machine length required for placement along the skew, add the rail-to-rail length and the extra end length from the following table using the plan specified skew rounded to the nearest 5 degrees. W is the rail-to-rail length as measured perpendicular to the centerline of the bridge.

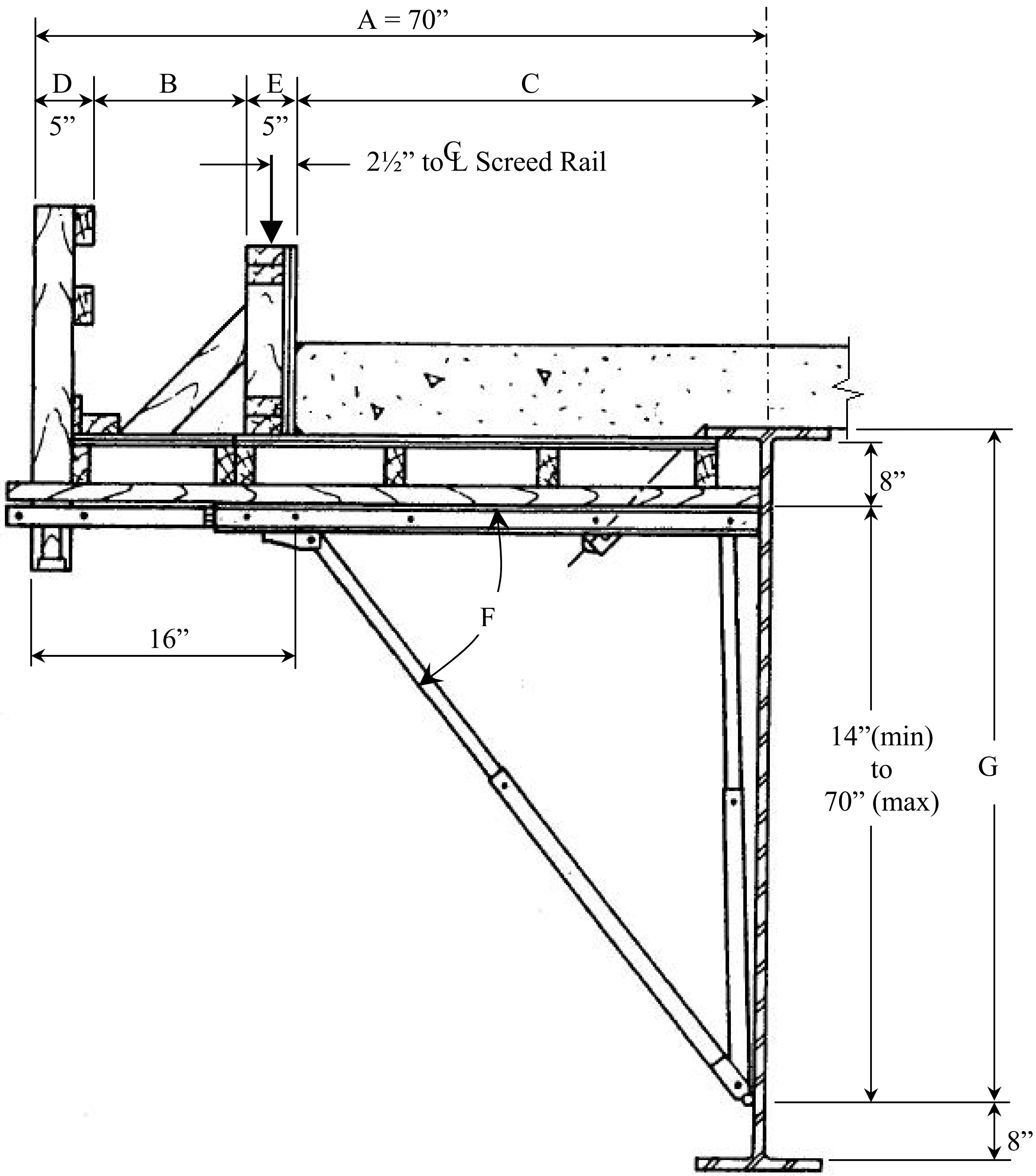
Skew Angle	Rail-to-Rail Length, ft.	Extra End Length, ft.
0	1.00 W	0.0
15	1.04 W	5.0
20	1.06 W	5.5
25	1.10 W	6.5
30	1.15 W	7.0
35	1.22 W	8.0
40	1.31 W	9.0
45	1.41 W	10.5
50	1.56 W	11.5
55	1.74 W	13.5

For total machine lengths of 36 ft. and less, assume a total machine weight of 7.6 kip. Add 0.09 kip for each additional foot of machine length required above 36 ft. The maximum total machine length shall not exceed 120 ft. If greater lengths are required, consult the Office of Structural Engineering for recommendations.

To determine the maximum wheel load, divide the total machine weight by 8.0.

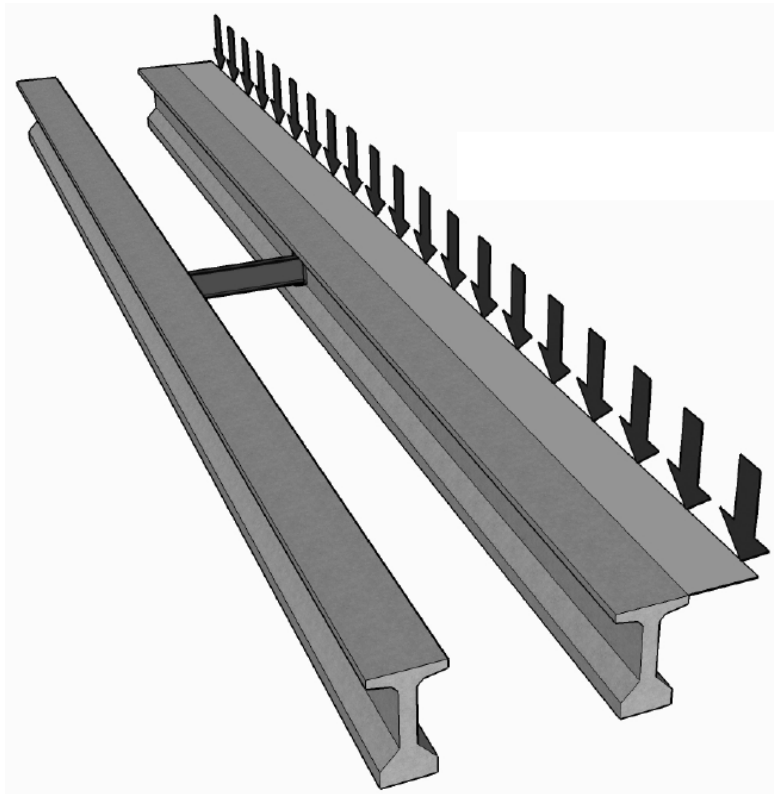
F. Bracket Data:

1. Refer to the following figure to determine TAEG dimensions A, B, C, D, E, F and G.
2. Designers may assume a center-to-center bracket spacing of 48.0 in.
3. Designers may assume a bracket weight of 50 lbs.



Assumptions for TAEG Bracket Data Input

For prestressed I-beam superstructures, Designers should verify that the intermediate crossframes/diaphragms in the exterior bay are capable of resisting the torsion caused by the cantilevered falsework.



302.2.7.3 DETERMINING EFFECT OF GIRDER TWIST

Once all sources of girder twist are quantified, Designers should determine the total effect that girder twist has on the finished deck surface. The primary effect of greatest concern is the loss of concrete cover over the top mat of deck reinforcing steel and the subsequent loss of deck thickness. The maximum loss due to twisting shall not exceed 0.5 in.

The total amount of girder twist at both the left and right screed rail should be determined as follows:

$\phi_{left} = (\phi_g + \phi_o + \phi_w)_{left}$ and $\phi_{right} = (\phi_g + \phi_o + \phi_w)_{right}$

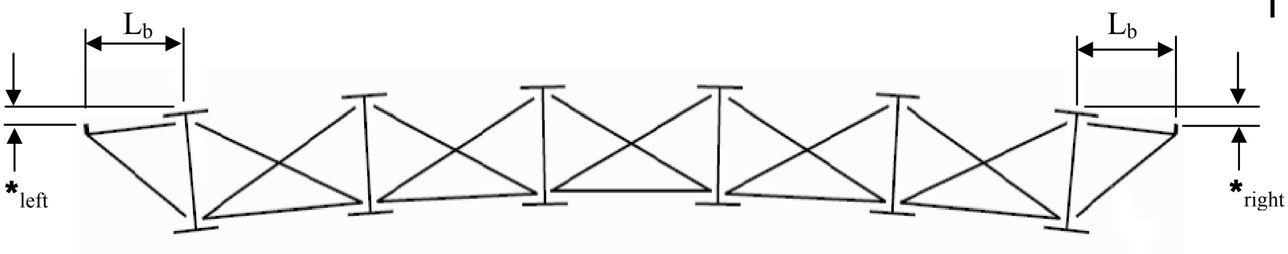
where:

$\mathbf{N}_g$ = Girder twist due to global superstructure distortion (See BDM Section 302.2.7.2.a)

$\mathbf{N}_o$ = Girder twist due to "oil-canning" (See BDM Section 302.2.7.2.b)

$\mathbf{N}_w$ = Girder twist due to girder warping (See BDM Section 302.2.7.2.c)

The total amount of screed rail deflection at both the left and right screed rail should be determined as follows:



$$\delta_{left} = \tan(\phi_{left}) \times L_b \text{ and } \delta_{right} = \tan(\phi_{right}) \times L_b$$

where:

$*_{left}$ $*_{right}$ = Deflection of the screed rail due to total girder twist (in.). Upward deflection is positive and downward deflection is negative.

L_b = Lateral distance from center of screed rail to centerline of fascia girder (in.)

The total loss of deck thickness should be determined as follows:

$*_{Total} = (*_{left} + *_{right})/2$

302.2.8 SLAB DEPTH OF CURVED BRIDGES

For a curved deck on straight steel beams, steel girders or prestressed I-beams, the distance from the top of the slab to the top of the beams or girders will vary from end to end. The slab depth dimension shall show this variation by giving the maximum and minimum depth dimensions with their respective location, over the piers, center of span, etc.

An alternate is to accommodate the differential depth by including it in the Camber Table as geometric camber.

302.2.9 STAGED CONSTRUCTION

For all bridge types, except non-composite concrete box beams, where the differential dead load deflection between adjacent beams, girders or structural slabs is greater than ¼ inch [6 mm], a deck closure is required if the bridge is constructed in stages.

The closure pour between the stages shall be a minimum width of 30 inches [800 mm] but should be wide enough to accommodate the required reinforcing steel lap splices. In special cases, this distance may be reduced when mechanical reinforcing steel connectors are used (see Section 200). The mechanical connector system used shall be able to develop 125 percent of the full yield strength of the reinforcing steel as a minimum.

Intermediate cross frames and diaphragms shall not be permanently attached in the closure pour location until the concrete pours on both sides of the closure pour location have been completed.

The two construction joints created by the concrete closure pour should be sealed with High Molecular Weight Methacrylate (HMWM), 705.15. The sealing width shown in the plans should be 2'-0" [600 mm], centered on the construction joints.

Placement of the staged construction joints above beam flanges is not recommended. The preferred location is the positive moment regions of the cast-in-place concrete deck slab.

The designer shall provide plan notes on the stage construction details sheet that detail the sequence of construction.

302.3 CONTINUOUS OR SINGLE SPAN CONCRETE SLAB BRIDGES

Continuous reinforced concrete slab bridge design shall be in accordance with *LRFD 4.6.2.3*.

For simple span reinforced concrete slab bridges cast in place directly on concrete substructures, the effective span length shall be considered equal to the clear span plus 15" [380 mm].

The Designer shall include a final deck surface elevation table. Elevations shall be shown for all profile grade lines, curblines, crownlines, and phased construction lines for the full length of the bridge. Bearing points, quarter-span points and mid-span points shall be detailed as well as any additional points required to meet a maximum spacing between points of 30'-0" [10 m].

302.4 STRUCTURAL STEEL

302.4.1 GENERAL

Structural steel should be designed utilizing a composite section. Refer to *LRFD 6.10.10.1* and BDM Section 1006 for more information.

A non-composite design may be used only if the design is the most economical.

The laterally unsupported length of top flanges of beam and girder members with a concrete deck encasing the top flange or compositely designed with studs shall be considered to be zero. In the absence of such fastening or direct contact of an individual beam or girder member, the unsupported length shall be considered as the distance between the diaphragms, struts, bridging, or other bracing.

For designs that assume the unbraced length of the top flange to be zero as mentioned above, the designer shall investigate the strength of the non-composite section during steel erection, deck slab construction, etc. using laterally unsupported lengths that reflect actual bracing conditions.

302.4.1.1 MATERIAL REQUIREMENTS

Types of steel to be selected for use in the design and construction of bridges is as follows:

A. ASTM A709[M] grade 50W shall be specified for an un-coated weathering steel bridge.

B. ASTM A709[M] grade 50 shall be specified for a coated steel bridge.

C. ASTM A709[M] grade 36 is not recommended and is being discontinued by the steel mills.

D. High Performance Steel (HPS), A709[M] grade 70W, un-coated weathering steel is most economical when used in the flanges of hybrid girders. Consult the Office of Structural Engineering for recommendations prior to specifying its use. A plan note is provided in the appendix.

The designer should recognize that "FULL BEARING" of beams and girders is not defined by AASHTO. "FULL BEARING" has been generally defined by ODOT as 75 percent of the bearing surface in contact and the other 25 percent with no gap greater than 1/32 inch [0.8 mm]. The designer shall specify the required fit definition when designing in conformance to the AASHTO design requirements for bolted splices in compression members.

Refer to Figure 302.4.1.14-1 for additional bolted splice details.

302.4.1.14.a BOLTS

Field splices in beams and girders shall be bolted connections using high strength bolts, ASTM A325[M].

The designer shall specify the diameter of the bolts and check that the type (Type I for Galvanized or Type III for Weathering) of A325[M] bolts is described in the coating notes or bolt material specifications.

Coating systems that are zinc based, such as OZEU, IZEU, Galvanizing or Metallizing require galvanized Type I bolts.

Un-coated weathering steel structures shall have A325[M], Type III bolts. If the faying surfaces under both the head and nut of every bolt of a weathering steel member are coated, specify galvanized A325[M] Type I bolts. Otherwise, specify A325[M], Type III bolts.

Generally, bolted splices should be designed using 1 inch [25 mm] or 1⅛ inch [29 mm] diameter bolts. No metric bolts or studs are available in the small quantities required for bridges.

302.4.1.14.b EDGE DISTANCES

The edge distances provided in *LRFD 6.13.2.6.6* are absolute minimums allowed during fabrication. For design and detailing purposes, 0.25 in. shall be added to the minimum edge distances listed in *Table 6.13.2.6.6-1.*

This increase will allow for fabrication tolerances when drilling bolt holes in splice plates, especially the inside flange splice plates.

302.4.1.14.c LOCATION OF FIELD SPLICES

Generally bolted splices should be located at points of dead load contraflexure on a continuous structure. Splices may also be supplied to help meet shipping and handling limitations. Plans should show optional field splice locations.

302.4.1.15 SHEAR CONNECTORS

Design shear connectors in accordance with *LRFD 6.10.10*.

Shear connectors shall be automatic end-welded stud-type. The use of channel sections is not

(THIS PAGE INTENTIONALLY LEFT BLANK)

allowed. 7/8 inch [22 mm] diameter studs are recommended as a standard diameter. The length of stud specified should be checked with manufacturers as to availability.

The Department's policy of using a 2 inch [50 mm] deep haunch over the top flange will have an effect on the length of shear studs.

Shear studs shall be field installed. In the case of galvanized structures, the design plans shall allow shop installation of studs prior to galvanizing or field installation after removing the coating by grinding at each stud location. If the studs are shop installed, the Contractor will be responsible for meeting all applicable OSHA requirements. A Detail note is available in Section 700.

302.4.2 ROLLED BEAMS

Effective in January 2006, the producers of rolled beams implemented changes to the physical dimensions of the W36X16 group of shapes (i.e. beams with 16" and wider flanges). The traditional W36X16 series of shape sizes will no longer be available from the producers. Below is a complete list for the new W36X16 group of shapes.

Designation	Area, *A* (in^2)	Depth, *d* (in)	Flange		Web
			Width, b_f (in)	Thickness, t_f (in)	Thickness, t_w (in)
W36 X 800	236.4	42.55	17.990	4.290	2.380
W36 X 652	192.5	41.05	17.575	3.540	1.970
W36 X 529	156.1	39.79	17.220	2.910	1.610
W36 X 487	143.8	39.33	17.105	2.680	1.500
W36 X 441	130.2	38.85	16.965	2.440	1.360
W36 X 395	117.4	38.41	16.830	2.200	1.220
W36 X 361	106.5	37.99	16.730	2.010	1.120
W36 X 330	97.4	37.67	16.630	1.850	1.020
W36 X 302	89.3	37.33	16.655	1.680	0.945
W36 X 282	83.4	37.11	16.595	1.570	0.885
W36 X 262	77.4	36.85	16.550	1.440	0.840
W36 X 247	72.9	36.67	16.510	1.350	0.800
W36 X 231	68.5	36.49	16.470	1.260	0.760

302.4.2.1 GALVANIZED BEAM STRUCTURES

If a galvanized bridge structure is the selected structure type, the following problems should be recognized and dealt with by the designer.

Galvanizing tanks are shallow and normally not longer than 45 feet [13.7 meters] in length. Therefore, beam lengths should not be longer than 60 feet [18.5 meters]. Before a design is necessary to avoid settlement due to group action by increasing the periphery of the soil mass.

In order to avoid the effects of downdrag, no battered piles shall be driven into new embankments until a waiting period for in-situ soil consolidation has concluded. Consult the Office of Structural Engineering for more information.

Abutment piles should be battered normal to the centerline of bearings.

Piles less than 15 ft [5 m] in length and driven to refusal on bedrock should not be battered.

A plan note is available in BDM Section 600 to establish the driving criteria for battered friction piles.

303.4.2.5 PILE DESIGN LOADS

Refer to BDM Section 202.2.3.2 for specific plan requirements.

Factored pile loads approaching the maximum factored structural resistance as specified in BDM Sections 202.2.3.2.a or the maximum Ultimate Bearing Value as specified in BDM 202.2.3.2.b should be utilized to minimize the number of piles.

303.4.2.6 PILES, STATIC LOAD TEST

The Designer shall specify a Static Load Test when the total pile order length for an individual structure exceeds 10,000 ft [3000 m] for piling of the same size and Ultimate Bearing Value. Static load testing is not required for piling driven to refusal on bedrock.

The Designer shall specify one subsequent static load test for each additional 10,000 ft [3000 m] increment of pile order length. Each static load test requires two dynamic testing items and two restrike items. Restrikes are a useful tool to determine if a driven pile gains or loses capacity over time.

The results of both the static and dynamic testing shall be forwarded to the Office of Structural Engineering to the attention of the Foundations Engineer. Refer to Section 600 for a General Note to include in the plans.

303.4.2.7 PILES, DYNAMIC LOAD TEST

The Department now requires dynamic load testing to establish the driving criteria (i.e. blow count) for all piling not driven to refusal on bedrock. The dynamic testing and resulting wave analysis has replaced the Engineering News Record Formula, used in previous issues of the CMS.

For an individual structure, the Designer shall specify one dynamic load testing item for each pile size. If multiple pile capacities are required for a given pile size, the Designer shall specify one testing item for each Ultimate Bearing Value. When static load tests are required, provide two dynamic load testing items and two restrike items for each static load test item.

The driving criteria for battered piles will be determined in the field as a function of a dynamically tested vertical pile of the same Ultimate Bearing Value. When battered piles are specified, refer to Section 600 for a General Note to include in the plans.

One dynamic load testing item consists of testing a minimum of 2 piles and performing a CAPWAP analysis on one of the two piles. One restrike item consists of performing dynamic testing on two piles and performing CAPWAP analysis on one of the two piles.

303.4.2.8 PILE FOUNDATION – DESIGN EXAMPLE

The following example for a 6-span bridge shall be used as a guide for specifying pile testing and estimated quantities for pile foundations.

Rear Abutment ~

30 - 12" C.I.P. Reinforced Concrete Piles

20 piles installed vertical & 10 piles battered

Ultimate Bearing Value = 152 kip

Estimated Length = 65 ft

Order Length = 70 ft (Total Length = 2100 ft)

Requires 1 dynamic load-testing item.

Piers 1, 2, 3, & 4 ~

80 - 14" C.I.P. Reinforced Concrete Piles at each pier

56 piles installed vertical & 24 piles battered

Ultimate Bearing Value = 250 kip

Estimated Length = 70 ft

Order Length = 75 ft (Total Length = 24,000 ft)

The total length (24,000 ft) requires 1 static load test item and 1 subsequent static load test. Each static load test requires 2 dynamic load testing items and 2 restrike items.

Pier 5 ~

52 - 14" C.I.P. Reinforced Concrete Piles

36 piles installed vertical & 16 piles battered

Ultimate Bearing Value = 270 kip

Estimated Length = 85 ft

Order Length = 90 ft (Total Length = 4680 ft)

The difference in Ultimate Bearing Value between piers 1, 2, 3 & 4 and pier 5 requires 1 dynamic testing item.

Forward Abutment ~

30 - 12" C.I.P. Reinforced Concrete Piles

20 piles installed vertical & 10 piles battered

Ultimate Bearing Value = 152 kip

Estimated Length = 75 ft

Order Length = 80 ft (Total Length = 2400 ft)

No additional dynamic load testing items are required.

For this example, the Designer should include notes **[606.2-1]**, **[606.2-4]** and **[606.2-5]** from Section 606.2 in the General Notes. Note **[606.2-1]** should be modified as follows:

PILE DESIGN LOADS (ULTIMATE BEARING VALUE): The Ultimate Bearing Value is 152 kip per pile for the rear and forward abutment piles. The Ultimate Bearing Value is 250 kip per pile for Pier 1, 2, 3, and 4 piles and 270 kip per pile for Pier 5 piles.

Abutment Piles:

30 piles 70 ft long, order length (Rear)

30 piles 80 ft long, order length (Forward)

1 dynamic load testing item

Pier 1, 2, 3, and 4 Piles:

320 piles 75 ft long, order length

1 static load test item

1 subsequent static load test item

4 dynamic load-testing items

4 restrike items

Pier 5 Piles:

52 piles 90 ft long, order length

1 dynamic load testing item

The Designer should provide the following items in the Estimated Quantities:

Item	Extension	Total	Unit	Description
506	11100	Lump	Sum	Static Load Test
506	12200	1	Each	Subsequent Static Load Test
507	00500	4200	ft	12" Cast-In-Place Reinforced Concrete Piles, Driven
507	00550	4500	ft	12" Cast-In-Place Reinforced Concrete Piles, Furnished
507	00600	26,820	ft	14" Cast-In-Place Reinforced Concrete Piles, Driven
507	00650	28,680	ft	14" Cast-In-Place Reinforced Concrete Piles, Furnished
523	20000	6	Each	Dynamic Load Testing
523	20500	4	Each	Restrike

303.4.3 DRILLED SHAFTS

To allow for the misalignment of drilled shafts that support single pier columns, the shaft diameter shall be 6 in. [150 mm] larger than the column diameter. To allow for misalignment of shafts into footings, footing widths shall be at least 1'-0" [305 mm] larger than the shaft diameter.

The diameter of bedrock sockets for drilled shafts are generally 6 in. [150 mm] less than the diameter of the shaft above the bedrock elevation. This downsize provides sufficient room is the shaft for the rock core barrel. Reinforcing steel cages should be based on the bedrock socket diameter.

For un-cased or temporarily cased drilled shafts, the spiral reinforcement should be a #4 [#13M] bar with a 4½ in. [115 mm] pitch. (Note: the above requirement shall be met even if the 4½ in. [115 mm] pitch may not meet the spiral requirements of *LRFD 5.7.4.6*) For shaft diameters 4.0 ft. and less, the out-to-out spiral diameter shall be 6 in. [150 mm] less than the rock socket diameter. For shaft diameters greater than 4.0 ft., the out-to-out spiral diameter shall be 12 in. [300 mm] less than the rock socket diameter. When steel casing is left in place, the spiral reinforcing pitch shall be 12 in. [300 mm].

The minimum clear distance between longitudinal reinforcement shall not be less than 3 times the bar diameter nor 3 times the maximum aggregate size. If bars are bundled in forming the reinforcing cage, the minimum clear distance between longitudinal reinforcement shall not be less than 3 times the diameter of the bundled bars. Where heavy reinforcement is required, consideration may be given to an inner and outer reinforcing cage.

For record and project use, each drilled shaft for a structure shall be individually identified by a unique number. The designer may choose to number the drilled shafts on the individual

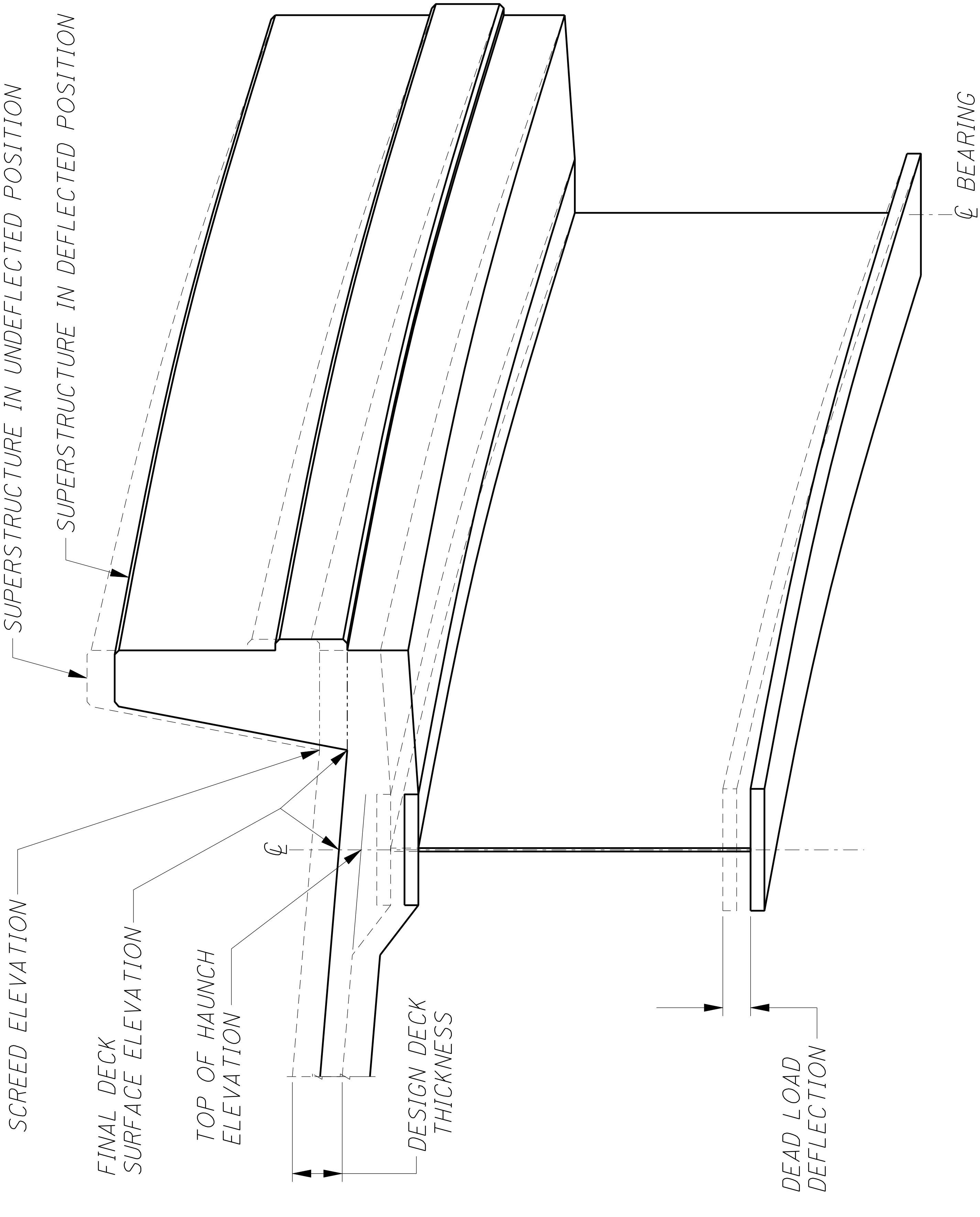


Figure 302.2.3-1

(THIS PAGE INTENTIONALLY LEFT BLANK)

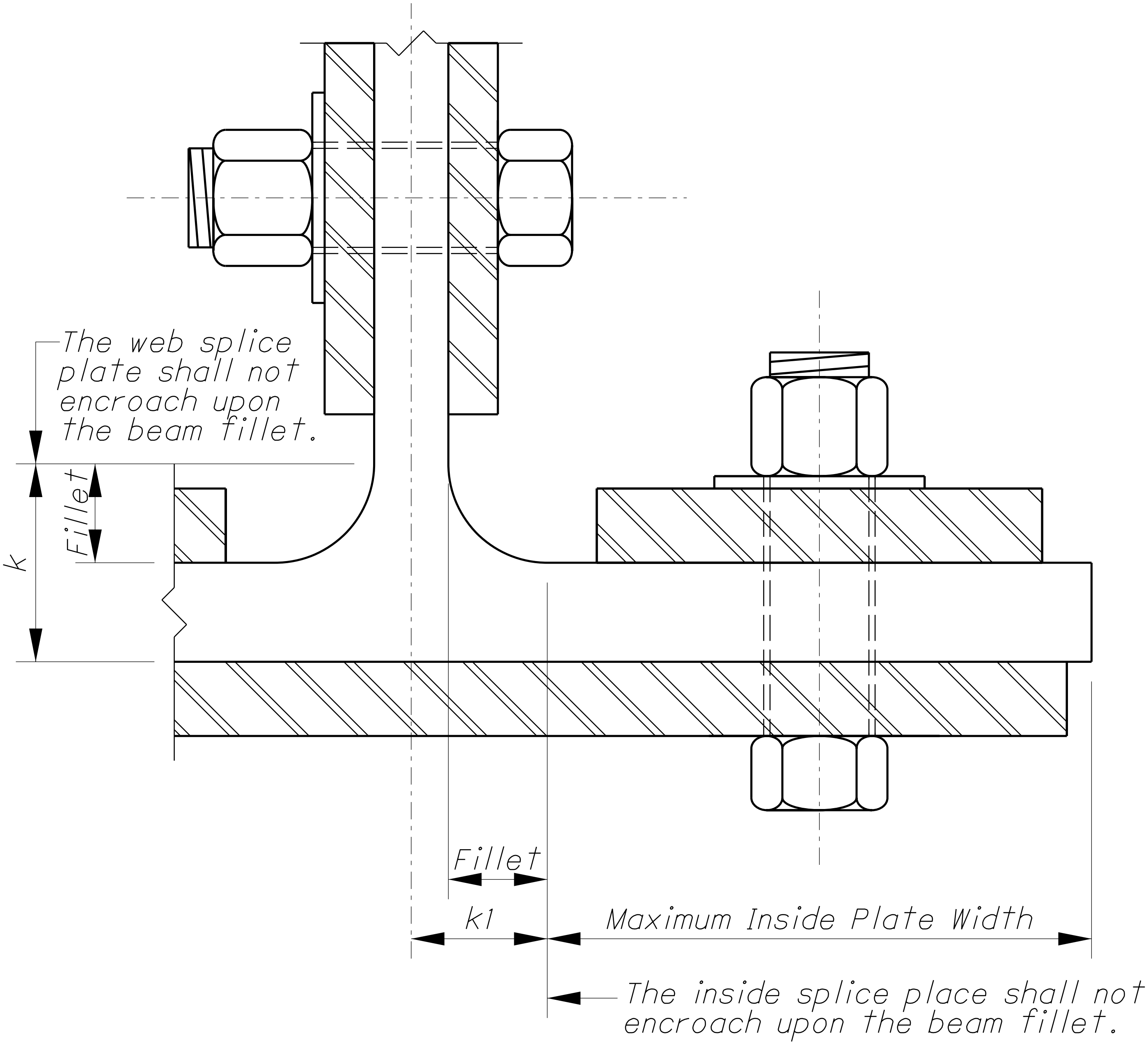


Figure 302.4.1.14-1

(THIS PAGE INTENTIONALLY LEFT BLANK)

(THIS PAGE INTENTIONALLY LEFT BLANK)

602.1 LRFD LOAD MODIFIERS

For bridges with non-redundant components, the following note shall be included:

[602.1-1] REDUNDANCY: The following item(s) were considered non-redundant for design and include a load modifier equal to 1.05 in accordance with the AASHTO LRFD Bridge Design Specifications, Article 1.3.4:

NOTE TO DESIGNER:
Include a list of all items considered non-redundant for design in accordance with BDM Section S*1.3.4*.

For bridges with non-redundant foundation components, the following notes shall be included:

[602.1-2] REDUNDANCY: The piles supporting the following substructure(s) were considered non-redundant for design and include a modified resistance factor equal to __(1)__ in accordance with the AASHTO LRFD Bridge Design Specifications, Article 10.5.5.2.3:

[602.1-3] REDUNDANCY: The drilled shafts supporting the following substructure(s) were considered non-redundant for design and include a modified resistance factor equal to __(1)__ in accordance with the AASHTO LRFD Bridge Design Specifications, Article 10.5.5.2.4:

NOTE TO DESIGNER:
Include a list of all substructures with pile foundations or drilled shafts considered non-redundant for design in accordance with *AASHTO LRFD 10.5.5.2.3* & *10.5.5.2.4*.

(1) Provide the modified resistance factor value. This should be equal to 80% of the resistance factor used for design on redundant pile foundations.

For all bridges the following note shall be included:

[602.1-4] OPERATIONAL IMPORTANCE: A load modifier of ___ has been assumed for the design of this structure in accordance with the AASHTO LRFD Bridge Design Specifications, Article 1.3.5 and the ODOT Bridge Design Manual, 2007.

NOTE TO DESIGNER:
Refer to BDM Section S*1.3.5* for guidance.

602.2 DESIGN LOADING

For bridges designed for highway loads, the design loading shall be:

[602.2-1] DESIGN LOADING: HL-93

Future Wearing Surface (FWS) of 0.060 kips/ft ^{2}.

For bikeway/pedestrian bridges that will not accommodate vehicular traffic the design loading shall be:

[602.2-2] DESIGN LOADING**:** 0.085 kips/ ft^2

For bikeway/pedestrian bridges subject to vehicular traffic the design loading shall be:

[602.2-3] DESIGN LOADING**:** 0.085 kips/ft^2 and H15-44 vehicle

602.3 DESIGN STRESSES

A. General Design Data:

[602.3-1] DESIGN DATA :

Concrete Class (1) - compressive strength 4.5 ksi (superstructure)

Concrete Class (2) - compressive strength 4.0 ksi (substructure)

Concrete Class S Modified - compressive strength 4.0 ksi (drilled shaft)

Reinforcing steel - minimum yield strength 60 ksi

Structural Steel - ASTM A709 Grade (3) - yield strength (3) ksi

Steel H-piles - ASTM A572 - yield strength 50 ksi

NOTE TO DESIGNER:

Modify note **[602.3-1]** as necessary. Delete references that are not applicable to project.

(1) Class S, Class HP or Class QSC2 Concrete for superstructure

(2) Class C, Class HP or Class QSC1 Concrete for substructure

(3) Grade 50 - yield strength 50 ksi, or

Grade 50W - yield strength 50 ksi, or

Grade HPS70W - yield strength 70 ksi, or

Grade 36 - yield strength 36 ksi

If more than one grade of steel is selected, the description shall clearly indicate where the different grades are used in the structure.

B. Additional Design Data for Prestressed Concrete Members:

Provide the following note in addition to note **[602.2-1]**.

expansion bearings only. Longitudinally applied superstructure loads are assumed to be transferred to the substructure as a friction loads (FR) equal to the nominal frictional resistances supplied by the bearings (see BDM Section 301.4.5). This assumption does not apply to fixed bearings. For fixed bearings, provide revised versions of these notes that list all applicable longitudinally applied superstructure loads transferred to the substructure through the bearing connections.

606 FOUNDATIONS

606.1 PILES DRIVEN TO BEDROCK

The following note generally will apply where steel-H piles are to be driven to bedrock:

[606.1-1] PILES TO BEDROCK: Drive piles to refusal on bedrock. The Department will consider refusal to be obtained by penetrating weak bedrock for several inches to a minimum resistance of 20 blows per inch or by contacting strong bedrock and the pile receiving at least 20 blows. Select the hammer size to achieve the required depth to bedrock and refusal. Instead of driving to refusal, the Contractor may perform dynamic load testing according to C&MS 523 to establish a driving criteria for each pile type and capacity. Establish the driving criteria to achieve an Ultimate Bearing Value that is 1.5 times the total factored load given below for the piles. Payment for dynamic load testing performed at the Contractor's option is included in the unit price pay item for piles driven.

The total factored load is __(1)__ kips per pile for the __(2)__ abutment piles. The total factored load is __(1)__ kips per pile for the __(2)__ pier piles.

Abutment piles:

__(3)__ piles __(4)__ feet long, order length

Pier piles:

__(3)__ piles __(4)__ feet long, order length

NOTE TO DESIGNER:

(1) Specify the total factored load according to BDM Section 202.2.3.2.a.

(2) Specify the location of piles for each total factored load.

(3) Specify the size of pile (e.g. HP 10 x 42 or 12 inch diameter).

(4) Specify the order length according to BDM Section 202.2.3.2.a and 303.4.2.1.

The following note, modified to fit the conditions, will apply where piles are located within a waterway and the scour depth is significant.

[606.1-2] PILES TO BEDROCK: Drive piles to refusal on bedrock. The Department will consider refusal to be obtained by penetrating weak bedrock for several inches to a minimum resistance of 20 blows per inch or by contacting strong bedrock and the pile receiving at least 20 blows. Select the hammer size to achieve the required depth to bedrock and refusal. Instead of driving to refusal, the Contractor may perform dynamic load testing according to C&MS 523 to establish a driving criteria for each pile type and capacity. Establish the driving criteria to achieve an Ultimate Bearing Value that is 1.5 times the total factored load given below for the piles. Payment for dynamic load testing performed at the Contractor's option is included in the unit price pay item for piles driven.

The total factored load is __(1)__ kips per pile for the __(2)__ abutment piles. The abutment piles were designed to accommodate __(3)__ ft. of scour. The total factored load is __(1)__ kips per pile for the __(2)__ pier piles. The pier piles were designed to accommodate __(3)__ ft. of scour.

Abutment piles:

__(4)__ piles __(5)__ feet long, order length

Pier piles:

__(4)__ piles __(5)__ feet long, order length

NOTE TO DESIGNER:

(1) Specify the total factored load according to BDM Section 202.2.3.2.a.

(2) Specify the location of piles for each total factored load.

(3) Specify the depth of anticipated scour.

(4) Specify the size of pile (e.g. HP 10 x 42 or 12 inch diameter).

(5) Specify the order length according to BDM Section 202.2.3.2.a and 303.4.2.1.

The following note, modified to fit the conditions, will apply where downdrag loads on the piles are anticipated.

[606.1-3] PILES TO BEDROCK: Drive piles to refusal on bedrock. The Department will consider refusal to be obtained by penetrating weak bedrock for several inches to a minimum resistance of 20 blows per inch or by contacting strong bedrock and the pile receiving at least 20 blows. Select the hammer size to achieve the required depth to bedrock and refusal. Instead of driving to refusal, the Contractor may perform dynamic load testing according to C&MS 523 to establish a driving criteria for each pile type and capacity. Payment for dynamic load testing performed at the Contractor's option is included in the unit price pay item for piles driven.

The total factored load is __(1)__ kips per pile for the __(2)__ abutment piles. The abutment piles include an additional __(3)__ kips of factored load per pile to account for possible downdrag loading. The total factored load is ____(1)____ kips per pile for the ____(2)____ pier piles. If performing dynamic load testing to establish driving criteria, the Ultimate Bearing Value is ____(4)____ kips per pile for the abutment piles and ____(4)____ kips per pile for the pier piles.

Abutment piles:

____(5)____ piles ____(6)____ feet long, order length

Pier piles:

____(5)____ piles ____(6)____ feet long, order length

NOTE TO DESIGNER:

(1) Specify the total factored load according to BDM Section 202.2.3.2.a.

(2) Specify the location of piles for each total factored load.

(3) Specify the anticipated factored downdrag loading.

(THIS PAGE INTENTIONALLY LEFT BLANK)

(4) Specify the Ultimate Bearing Value for dynamic load testing, including downdrag.

(5) Specify the size of pile (e.g. HP 10 x 42 or 12 inch diameter).

(6) Specify the order length according to BDM Section 202.2.3.2.a and 303.4.2.1.

606.2 FRICTION TYPE PILES

The following notes, modified to fit the specific conditions for the foundation required, will apply in all cases except where the piles are to be driven to bedrock. Provide the actual calculated Ultimate Bearing Value as shown below:

[606.2-1] PILE DESIGN LOADS (ULTIMATE BEARING VALUE): The Ultimate Bearing Value is _(1)_ kips per pile for the _(2)_ abutment piles. The Ultimate Bearing Value is _(1)_ kips per pile for the _(2)_pier piles.

Abutment piles:

_(3)_ piles _(4)_ feet long, order length

_(5)_ Dynamic load testing items

Pier piles:

_(3)_ piles _(4)_ feet long, order length

_(5)_ Dynamic load testing items

NOTE TO DESIGNER:

(1) Specify the Ultimate Bearing Value according to BDM Section 202.2.3.2.b.

(2) Specify the location of piles for each Ultimate Bearing Value.

(3) Specify the size of pile (e.g. HP 10 x 42 or 12 inch diameter).

(4) Specify the order length according to BDM Section 202.2.3.2.b and 303.4.2.1.

(5) Specify the number of dynamic load testing items according to BDM Section 303.4.2.7.

The following note, modified to fit the conditions, will apply where piles are located within a waterway and the scour is anticipated.

[606.2-2] PILE DESIGN LOADS (ULTIMATE BEARING VALUE): The Ultimate Bearing Value is _(1)_ kips per pile for the _(2)_ abutment piles. The Ultimate Bearing Value is _(1)_ kips per pile for the pier piles. The pier piles include an additional _(3)_ kips per pile of Ultimate Bearing Value due to the possibility of losing _(7)_ ft. of frictional resistance due to scour.

Abutment piles:

_(4)_ piles _(5)_ feet long, order length

_(6)_ Dynamic load testing items

Pier piles:

_(4)_ piles _(5)_ feet [meter] long, order length

_(6)_ Dynamic load testing items

NOTE TO DESIGNER:

(1) Specify the Ultimate Bearing Value according to BDM Section 202.2.3.2.h.

(2) Specify the location of piles for each Ultimate Bearing Value.

(3) Specify the additional amount of Ultimate Bearing Value according to BDM Section 202.2.3.2.h.

(4) Specify the size of pile (e.g. HP 10 x 42 or 12 inch diameter).

(5) Specify the order length according to BDM Section 202.2.3.2.h and 303.4.2.1.

(6) Specify the number of dynamic load testing items according to BDM Section 303.4.2.7.

(7) Specify the scour depth.

The following note, modified to fit the conditions, will apply where downdrag loads on the piles are anticipated.

[606.2-3] PILE DESIGN LOADS (ULTIMATE BEARING VALUE): The Ultimate Bearing Value is _(1)_ kips per pile for the _(2)_ abutment piles. The Ultimate Bearing Value is _(1)_ kips per pile for the _(2)_ pier piles. The addition of _(3)_ kips of Ultimate Bearing Value per abutment pile is due to possible downdrag loads caused by settlement and to account for side friction within the downdrag zone that must be overcome during pile driving.

Abutment piles:

_(4)_ piles _(5)_ feet long, order length

_(6)_ Dynamic load testing items

Pier piles:

_(4)_ piles _(5)_ feet long, order length

_(6)_ Dynamic load testing items

NOTE TO DESIGNER:

(1) Specify the Ultimate Bearing Value according to BDM Section 202.2.3.2.c.

(2) Specify the location of piles for each Ultimate Bearing Value.

(3) Specify the additional amount of Ultimate Bearing Value according to BDM Section 202.2.3.2.c. This amount includes the factored downdrag load and the unfactored side resistance from the soil in the downdrag zone.

(4) Specify the size of pile (e.g. HP 10 x 42 or 12 inch diameter).

(5) Specify the order length according to BDM Section 202.2.3.2.c and 303.4.2.1.

(6) Specify the number of dynamic load testing items according to BDM Section 303.4.2.7.

Provide the following note when Static Load Testing is required according to Section 303.4.2.5. Modify the note as necessary to fit the specific condition.

[606.2-4] STATIC LOAD TEST: Perform dynamic testing on the first two production piles to determine the required blow count for the specified Ultimate Bearing Value. Perform the static load test on either pile. Do not over-drive the selected pile. Drive the third and fourth production piles to 75% and 85% of the determined blow count, respectively and perform dynamic testing on each. The test piles and the reduced capacity piles shall not be battered. After installation of the first four production piles, cease all driving operations on piling represented by the static load testing for a minimum of 7 days. After the waiting period, perform pile restrikes on the four piles (two restrike test items). The Engineer will review the results of the pile restrikes and establish the driving criteria for the remaining piling represented by the testing. Submit all test results to the Office of Structural Engineering.

For subsequent static load tests, upon completion of a 10,000 ft increment of driven length, repeat the above procedure for the initial static load test. If necessary, the Engineer will revise the driving criteria for the remaining piling accordingly.

When performing the restrike, if the pile has not reached the blow count determined for the plan specified Ultimate Bearing Value, continue driving the pile until this capacity is achieved.

Provide the following note when battered friction piles are specified.

[606.2-5] BATTERED PILES: The blow count for battered piles shall be the blow count determined for vertical piles of the same Ultimate Bearing Value divided by an efficiency factor (D). Compute the efficiency factor (D) as follows:

$$D = \frac{1 - UG}{\sqrt{\left(1 + G^2\right)}}$$

U = Coefficient of friction, which is estimated at 0.05 for double-acting air operated or diesel hammers; 0.1 for single-acting air operated or diesel hammers; and 0.2 for drop hammers.

G = Rate of batter (1/3, 1/4, etc.)

The following note, modified to fit the specific conditions for the foundation required, will apply when uplift loads control the design of the pile. In this case, the piles are typically driven to a pile tip elevation and dynamic load testing of the pile is not performed.

[606.2-6] PILES DRIVEN TO TIP ELEVATION FOR UPLIFT: Drive the piles to the pile tip elevation shown on the plans. Do not perform dynamic load testing on piles driven to a tip elevation. Select the hammer size to achieve the required depth. Provide plain cylindrical casings with a minimum pile wall thickness of (1) inch for piles driven to a tip elevation.

Abutment piles:

(2) piles (3) feet long, order length

NOTE TO DESIGNER:

(1) Specify the minimum pile wall thickness for cast-in-place reinforced concrete piles. Determine the minimum pile wall thickness from a pile drivability analysis. Remove this sentence if the piles are H-piles.

(2) Specify the size of pile (e.g. HP 10 x 42 or 12 inch diameter).

(3) Specify the order length according to BDM Section 202.2.3.2.b and 303.4.2.1.

606.3 STEEL PILE POINTS

Use the following note where steel points are required, and see Section 202.2.3.2.a.

[606.3-1] ITEM 507, STEEL POINTS, AS PER PLAN: Use steel pile points to protect the tips of the proposed steel “H” piling. Furnish steel points from the following manufactures/suppliers: Associated Pile and Fitting Corporation, 262 Rutherford Blvd., Clifton, New Jersey 07014, phone: (973)773-8400, (800)526-9047, fax: (973)773-8442; International Construction Equipment, Inc., 301 Warehouse Drive, Matthews, North Carolina 28015, phone: (704)821-8200, (888)423-8721, fax: (704)821-8201; Dougherty Foundation Products, Inc., P.O. Box 688, Franklin Lakes, New Jersey 07417, phone: (201)337-5748, fax: (201)337-9022; Versa Steel Inc., 1618 N.E. First Ave., Portland, Oregon 97232, phone: (503)287-9822, (800)678-0814, fax: (503)287-7483; Versabite Piling Accessories, 1704 Tower Industrial Dr., Monroe, North Carolina 28110, phone: (800)280-9950, (704)225-1566, fax: (704)225-1567; or by a manufacturer that can furnish a steel point that is acceptable to Director. The material used for the manufacturing of pile points shall conform to ASTM A27/A27M 65/35 [450/240] – Class 2 – Heat Treated or AASHTO M103/M103M 65/35 [450/240] – Heat Treated. Weld the pile points to the pile in accordance with AWS D1.5 or the manufacturer’s written welding procedure supplied to the engineer before the welding is performed. Submit a notarized copy of the mill test report to the Engineer.

606.4 PILE SPLICES

Provide the following note when H-piles are specified.

610.6 COFFERDAMS, CRIBS AND SHEETING

Use this note when the plans include detail designs for temporary shoring.

[610.6-1] ITEM 503, COFFERDAMS, CRIBS, AND SHEETING, AS PER PLAN:
The design shown on the plans for temporary support of excavation is one representative design that may be used to construct the project. The Contractor may construct the design shown on the plans or prepare an alternate design to support the sides of excavations. If constructing an alternate design for temporary support of excavation, prepare and provide plans in accordance with C&MS 501.05. The Department will pay for the temporary support of excavation at the contract lump sum price for Cofferdams, Cribs, and Sheeting. No additional payment will be made for providing an alternate design.

610.7 DECK PLACEMENT NOTES

610.7.1 FALSEWORK AND FORMS

Use the following note when web depths greater than 84 in. are specified.

[610.7.1-1] ITEM 511, CLASS HP CONCRETE, SUPERSTRUCTURE, AS PER PLAN *

Locate the lower contact point of the overhang falsework at least _**_ inches ± 2 in. above the top of the girder's bottom flange. The bracket contact point location requirements of C&MS 508 do not apply.

NOTE TO DESIGNER:

* Modify the pay item description to fit the specific project requirements.

** The minimum dimension for the location for the lower point of contact should be 76 in. below the bottom of the top flange. Designers should verify the acceptability of the design within the range of tolerance specified.

610.7.2 DECK PLACEMENT DESIGN ASSUMPTIONS

Use the following note on all projects requiring mechanized finishing machines to place deck concrete.

[610.7.2] DECK PLACEMENT DESIGN ASSUMPTIONS:

The following assumptions of construction means and methods were made for the analysis and design of the superstructure. The Contractor is responsible for the design of the falsework support system within these parameters and will assume responsibility for superstructure analysis for deviation from these design assumptions.

An eight wheel finishing machine with a maximum wheel load of _____ kips for a total machine load of _____ kips.

A minimum out-to-out wheel spacing at each end of the machine of 103”.

A maximum spacing of overhang falsework brackets of 48 in.

A maximum distance from the centerline of the fascia girder to the face of the safety handrail of 65”.

NOTE TO DESIGNER:

Refer to BDM Section 302.2.7.2.c for design information regarding finishing machine loads.

(THIS PAGE INTENTIONALLY LEFT BLANK)

702.12 ERECTION BOLTS

Where erection bolts are specified for attaching crossframes on steel girder or rolled beam bridges, and the expected dead load differential deflection at each end of the crossframes is less than or equal to ½" [13 mm] provide the following note. (Do not use the note if standard drawing GSD-1-96 is being referenced.)

[702.12-1] ERECTION BOLTS: The hole diameter in the cross frames and girder stiffeners shall be 3/16" larger than the diameter of the erection bolts. Erection bolts shall be high strength bolts and shall remain in place. Supply two hardened washers with each high strength bolt. Fully torque the bolts or use a lock washer in addition to the two hardened washers. Furnish erection bolts as part of Item 513.

[702.12-2] Note Retired – See Appendix

702.13 WELDED ATTACHMENTS

Provide the following note on plans for steel beam or girder bridges:

[702.13-1] WELD ATTACHMENT of supports for concrete deck finishing machine to areas of the fascia stringer flanges designated "Compression". Do not weld attachments to areas designated "Tension". Fillet welds to compression flanges shall be at least 1" from edge of flange, be no more than 2" long, and be at least 1/4" for thicknesses up to 3/4" or 5/16" for greater than 3/4" thick.

702.14 DECK ELEVATION TABLES

702.14.1 SCREED ELEVATION TABLES

Screed elevation tables are required for concrete decks on structural steel beam, structural steel girder, prestressed I-beam, composite box beam and other superstructure types with cast-in-place concrete decks. Screed elevations are not required for slab bridges. The general criteria for screed elevation tables are defined in BDM Section 302.2.3. Refer to Figure 0-1 for an example screed table for structural steel members and Figure 0-2 for an example screed table for composite box beams. In lieu of a table format, the designer may supply screed elevations through the use of a deck plan view showing elevations and stations of the points required in BDM Section 302.2.3.

In addition to the screed elevation table or diagram, provide a screed elevation note similar to the one below to define the elevations that are given. The screed elevation locations should be identified on the transverse section.

[702.14.1] SCREED ELEVATIONS shown represent the theoretical deck surface location prior to deflections caused by deck placement and other anticipated dead loads.

702.14.2 TOP OF HAUNCH ELEVATION TABLES

Top of haunch elevation tables are required for concrete decks on structural steel beam, structural steel girder, prestressed I-beam and other superstructure types requiring deck falsework. Top of haunch elevations are not required for slab bridges. The general criteria for top of haunch elevation tables are defined in BDM Section 302.2.3.

In addition to the top of haunch elevation table, provide a top of haunch elevation note similar to the one below to define the elevations that are given. The top of haunch elevation locations should be identified on the transverse section.

[702.14.2-1] TOP OF HAUNCH ELEVATIONS shown represent the theoretical location of the bottom of the deck above the beam/girder haunch prior to deflections caused by deck placement and other anticipated dead loads.

702.14.3 FINAL DECK SURFACE ELEVATION TABLES

Final deck surface elevation tables are required for concrete decks on structural steel beam, structural steel girder, prestressed I-beam, composite box beam and other superstructure types with cast-in-place concrete decks including slab bridges. The general criteria for final deck surface elevation tables are defined in BDM Section 302.2.3.

In addition to the final deck surface elevation table, provide a final deck surface elevation note similar to the one below to define the elevations that are given.

[702.14.3-1] FINAL DECK SURFACE ELEVATIONS shown represent the deck surface location after all anticipated dead load deflections have occurred.

702.15 ELASTOMERIC BEARING MATERIAL REQUIREMENTS

Use the following note for elastomeric bearings designed in accordance with *LRFD 14.7.6* (i.e. Method A)

[702.15-1] ELASTOMERIC BEARINGS: The elastomer shall have a hardness of ___ (50 or 60) durometer. The bearings were designed in accordance with Section 14.7.6 (Method A) of the AASHTO LRFD Bridge Design Specifications. The Long-term Compression Proof Load Test (AASHTO Standard Specifications for Highway Bridges, Division II, Section 18.7.2.6) is not required.

Use the following note for elastomeric bearings designed in accordance with *LRFD 14.7.5* (i.e. Method B)

[702.15-2] ELASTOMERIC BEARINGS: The elastomer shall have a hardness of ___ (50 or 60) durometer. The bearings were designed in accordance with Section 14.7.5

(THIS PAGE INTENTIONALLY LEFT BLANK)

STRUCTURAL STEEL SCREED TABLE								
	SPAN NUMBER (1)							
Point (2)	Station	Station	Station	Station	Station	Station	Station	Station
	Bearing Pt	1/4 Pt	Pt (3)	Mid Span	Pt (3)	Splice Pt	1/4 Pt	Bearing Pt
Left Curb Line								
Phased Const Line								
Centerline								
Right Curb Line								

(1) Detail all spans.

(2) Station all Points for screeds.

(3) Additional points required in a span if the distance between bearing points, 1/4 points, mid span and/or splice points exceeds 30 feet. If the distance does exceed 30 feet, locate the additional point midway between standard points.

Figure 702.14-1

COMPOSITE BOX BEAM SCREED TABLE							
	SPAN NUMBER (1)						
Point (2)	Station Bearing Pt	Station 1/4 Pt	Station Pt (3)	Station Mid Span	Station Pt (3)	Station 1/4 Pt	Station Bearing Pt
Left Curb line							
Phased Const Line							
Centerline							
Right Curb Line							

(1) Detail all spans.

(2) Station all Points for screeds.

(3) Additional points required in a span if the distance between bearing points, 1/4 points and/or mid span points exceeds 30 feet. If the distance does exceed 30 feet, locate the additional point midway between standard points.

Figure 702.14-2